DEVELOPMENT OF A RATIONAL PROCEDURE FOR PAVEMENT DESIGN

BY

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CERTIFICATION

This is to certify that this research project is prepared and submitted by **Mr. ADERINOLA, O. S.** (CVE/95/7019) and approved as to style and content by:

.... 17-04-99
(Supervisor) Date

DEDICATION

Dedicated to the glory of GOD and to the memory of my late father MR. P. A. IKUSEMORI who laboured so hard to sow but whom death disallowed from reaping the fruits of his labour. May his soul continue to rest in perfect peace – Amen.

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\- O.S. ADERINOLA

ABSTRACT

This study seeks to obtain an optimal pavement layout by combining theoretical and experimental field approaches to pavement design thereby side-tracking full-scale tests for each new road. To do this, a comprehensive data on traffic census, fundamental properties of road materials such as asphalt, laterites and stabilized laterites were collected from relevant Ministries, Journals, textbooks, laboratory test reports and engineering companies. A computer (FORTRAN) programme was written using BISAR programme equation in conjunction with Smith-Witczak (1981) equation. The results were used to obtain a mechanistic pavement design with inputs from American Association of State Highway and Transportation Officials (AASHTO) design guide. The approach produced an optimal pavement layout of 625mm (total thickness) that can exceed its design life with overloaded traffic. It also compares favourably with former procedures for tropical countries. Further research into this "theory-practical" approach is recommended.

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CHAPTER ONE

INTRODUCTION

1.1 GENERAL BACKGROUND OF ROADS AND ROADWAY PAVEMENTS

1.1.1 Roads

Roads can be considered as causes as well as consequence of civilization. As civilization is advanced and prosperity increases, there is always an inevitable demand for better and speedier communication facilities especially roads. Indeed it can truly be said that the prosperity of a nation is bound up with the state of its roads, and that the roads act as a palimpsest of a nation's history (O 'Flaherty, 1973).

Roads facilitate the movement of both people and goods from place to place and they also provide social and commercial interactions between communities, towns and even nations in some cases. Prior to the introduction of automobiles, there was no urgent need to design roads similar to modern roadways. Horse-drawn wagons and other slow moving vehicles did not require a paved road as long as the ground was hard enough for the vehicles to travel on. Although many roads were built in ancient times such as the Roman Appian Way and the express roads of the early Chinese dynasty, the purpose of the construction was to provide a hard surface for the repeated use without interrupting travel speed. Meanwhile, since World War II, the legal speed of vehicles has increased rapidly, from 64 to 96 kilometre per hour and even 128 kilometre per hour in some countries (Yang, 1972). As a result of this, many engineers began to realize the inadequacy of pavement design methods of those days and as such intensified research(es) into different methods of pavement design.

An example of such research is the American Association of States Highway and Transportation Officials (AASHTO) road test undertaken cooperatively by 49 of the states in the U.S.A, the District of Columbia, Puerto Rico, the Bureau of Public Roads and various industry groups. AASHTO operating committee on design released recommendations for design methods for both flexible and rigid pavements in 1961. In 1972, a revised interim design guide was published which became what is now popularly known as AASHTO design guide. (Aderinola, 1996).

1.1.2 Roadway Pavement

According to Salter (1979), a roadway pavement is a structure consisting of super-imposed layers of selected and processed materials whose primary function is to distribute the applied vehicle loads to the subgrade. There are two types of roadway pavements. These are:

(a) Flexible pavement;

(b) Rigid pavement.

a. Flexible Pavement: This consists of several courses of treated and/or untreated mineral materials placed upon the subgrade or prepared earthworks foundation. The quality (and therefore the cost) of the materials involved will usually increase with each successive course from the subgrade. These courses are basically:

(i) Subgrade; (ii) Sub-base

(iii) Base; and (iii) Surfacing.

i. Subgrade - This is the natural material lying under the grade line material often referred to as subgrade soil, base soil or foundation soil. The strength of the subgrade is a major factor in determining the thickness of the pavement and this is assessed on the California Bearing Ratio (CBR) scale. The top 15cm of the subgrade is sometimes improved by compaction or chemical stabilization.

ii. Subbase Course - It consists of an imported material of high quality than the subgrade, but the term "sub-base" usually indicates that it does not meet the quality requirement for a base course material. It is used in the pavement structure for reasons of economy as in the case of locally available granular materials. Sub-base materials may or may not be treated with additives.

iii. Base Course - It is placed directly on the prepared subgrade or upon the completed subbase if such a course is included in the pavement structure. Base course materials are comprised of naturally occuring mineral aggregates of a superior quality to those used in sub-base. Also, the materials may or may not be treated with additives. In recent years, the use of bituminous treated base course material has been gaining favour because of its economy binding and water-proofing qualities which improve the base-course durability (Beaty and Westall, 1971).

iv. Surfacing or Surface Course - This is the riding surface of a roadway pavement. It is placed on the binder course in case of asphalt concrete pavements. The surfacing materials are to provide the followings:-

1. a smooth quiet-riding surface;
2. a surface which is cohesive and durable to resist the wearing stresses imposed by traffic;
3. a surface with a high co-efficient of friction between the surface and the vehicle tyres so that adequate skid resistance is achieved;
4. a layer of material which is relatively impervious to infiltration of water and air thus preventing accelerated ageing of the bituminous binder material (Yang, 1972).

(b) Rigid Pavement: A rigid or concrete pavement consists of a concrete slab laid on a sub-base or base material and all rests on the subgrade. A major portion of the load-carrying capacity of rigid pavement is derived from the slab itself. This is often called or referred to as beam action. Stresses in this type of pavement result from wheel loads cyclic changes in temperature (warping and shrinkage or expansion), changes in moisture, and volumetric changes in the subgrade or base course (Yoder, 1959).

1.2 PAVEMENT DESIGN

Pavement design is a procedure that aims at providing a pavement structure that will serve traffic safely, conveniently and economically during the design life of the pavement (Gichaga and Parker, 1988). The parameters considered in the design of pavement are primarily the traffic loading, climate or environment, material characteristics, design period, cost, construction and maintenance. All these could also be called the elements of thickness design.

1.3 LATERITES AND STABILIZED LATERITIES

1.3.1 Laterites

Laterites constitute important materials for engineering uses. According to Evans (1910), a relatively large proportion of the world especially the tropical zone is covered with lateritic soils. Laterites are highly ferroginous deposits which were first discovered by Buchanan in Malabar during his journey through countries of Mysore, Canara and Malabar in 1800. There have been many definition of laterites but most of them have been based on the silica content. The most prominent definition is that by Martins and Doyne (1930), who based their definitions on the silica - alumina ratio. According to them, true laterites are earth materials which have silica - alumina ratio ($S_iO_2/A1_2O_3$) less than 1.33 and those with values between 1.33 to 2.0 were classified as lateritic soils while those with values above 2.0 were called non-lateritic soils.

1.3.2 Stabilized Laterites

Stabilized laterites are lateritic soils that had been subjected to every physical, physico-chemical and chemical methods to enable them serve better their intended engineering purposes (George, et al, 1992). They are lateritic soils whose properties had been improved in order to use them as roadway bases, sub-bases and occasionally, surface courses in highway engineering.

Stabilization reduces the sensitivity of lateritic soils to moisture changes and subsequent loss of strength. Mechanical stabilization improves the soils grading, usually by adding materials corresponding to the depleted grain-sizes in the original soil. Chemical stabilization of the soils (e.g. cement or lime) promotes a chemically induced increase in strength. Lime is sometimes used to reduce plasticity and to stabilize highly plastic soil before adding cement (Sherwood, 1968).

1.4 OBJECTIVES

The objectives of this project are:-

(a) To review the subject matter of civil engineering systems analysis as a tool for designs in any aspect of civil engineering.

(b) To review the subject matter of pavement design.

(c) To evolve a procedure for pavement design that optimizes materials usage and is customized to the local environment in Nigeria.

1.5 JUSTIFICATION

The cost of roads and pavements vary from year to year, from place to place and from job to job depending upon availability of materials, wage scales et. cetera (Babcock and Bone, 1962). As a result of global economic recession, and thus economic recovery programmes (for example, Structural Adjustment Programmes (SAP) adopted by some African countries, the cost of road construction or rehabilitation has become prohibitive. For this reasons, some of these countries had initiated researches into what could reduce this cost through design or construction of the roads. Ogweno-Bonyo (1997) reported a chemical solution, con Aid, being used in Uganda for road construction to minimize maintenance cost and to increase life-span of roads. Lamers (1984) suggested a thick bituminous coating of 6cm during construction and a further coating of 2 - 3cm after 15 years to prolong the life of the pavement. Ajayi (1960) suggested a Vehicle Damage Factor (VDF) of 4.5 for the design of Nigerian roads to forestall early failure of the roads.

To accomplish the aim of good and durable roadway pavements, the two approaches to pavement design - theoretical analysis approach and experimental field approach should be combined in a theoretical form so that the design and evaluation of a pavement structure for a new road can be done by direct computations without the necessity of carrying out full-scale tests for each new need. It is in view of this, a sensible order in which this could be done using stabilized lateritic soil is being sought. Ahlvin, et al (1974) tried the combination and got a good correlation between the observed performance of numerous full-scale accelerated traffic test pavements for a new airport and the maximum shearing strains computed for these pavements.

CHAPTER TWO

LITERATURE REVIEW

2.1 ROADWAY STRUCTURE

According to Haas and Hudson (1979), pavement is the "upper portion of the roadway, airport or parking lot structure" and includes all the layers resting on the subgrade. The pavement in addition, is considered to have a bound surface and includes "the load carrying capacity of the subgrade". Salter (1979), defined roadway pavement as a structure consisting of super-imposed layers of selected and processed materials whose primary function is to distribute the applied vehicle loads to the subgrade. The two types of pavements had been discussed in Chapter One.

2.2 PAVEMENT DESIGN METHODS

The primary objective of highway pavement design is to provide an acceptable roadway structure that can withstand deteriorating effects of traffic and environment for the service life of the facility. In addition, the pavement structure must adequately serve the demands of the road users at an acceptable level of performance. A properly designed, constructed and maintained pavement is a major factor in providing economical, efficient, safe, convenient and comfortable highway travel (Hejal et al, 1971).

In the present engineering practice in the design of pavement, the commonly used methods can be divided into two groups:-

- the theroretical analysis approach;
- the experimental field approach.

Much efforts has been made in both approaches, and in general, there is poor correlation between them. Theoretical works such as elastic layered systems and the finite-element methods of analysis, are designed to compute the stress, strain, and displacement in the pavement structure but are not capable of evaluating the performance of the pavement such as cracking, distortion, etc. that are of vital interest to the pavement design engineers. On the other hand, experimental full-scale test section approach produces reliable performance data, and such test sections are both expensive and time consuming, and provide only minimal amount of basic test data. There had been some attempts to combine the theory and field performance in a theoretical form so that the design and evaluation of a pavement structure for a new road can be accomplished by direct computations without the necessity of conducting full-scale tests for each new need.

Ahlvin, et al (1974) attempted the combination and got a good correlation between the observed performance of numerous full-scale accelerated traffic test pavements for a new airport and the maximum shearing strains computed for these pavements.

2.2.1 Theoretical Analysis Approach:

According to Gichaga and Parker (1988), theoretical pavement design involves the assumption of a pavement structure system. The strength characteristics (in terms of the modulus and poissons ratio) for each layer may be established or can be established. Traffic loading is then introduced and structural analysis carried out to determine the stresses and strains at critical points in the structure. The values of stresses and strains obtained from the analysis are compared with maximum allowable values, to determine whether the design is satisfactory, and if it is not, another system is tried (Gichaga and Parker, 1988).

In essence, theoretical or analytical design should involve the following steps:-

- development of mathematical model to represent pavement behaviour;
- selection of appropriate solution techniques to the equations developed above and thus the computation of stresses, strains and deformation at critical points in the pavement structure;
- material characterisation for the various layers of the pavement structure for a given environment;
- establishing design criteria such as in terms of allowable stresses or strains in certain critical points in the structure;

2.2.1.1 <u>Mathematical Model</u> - The mathematical models are the tools by which engineers apply scientific principles to the solution of engineering problems even without the benefit of past experience (Yang, 1972). A mathematical model consists of three submodels:-

i. the equilibrium of the pavement system under the influence of external loads;

ii. for a given support condition, an evaluation of the deformations and stresses in the pavement element;

iii. a characterization of the fundamental properties of pavement materials and their effect on the equilibrium and stability of the pavement structure.

The early development of mathematical models for pavement structures was confined primarily to the second submodel. In 1884, Hertz proposed a mathematical method for analysing an elastic slab. Westergaard (1926), simplified the mathematical manipulation for application to practical design problems. Pickett-Ray (1951) introduced the use of influence charts and thus reduced the design computation to almost nothing. The Westergaard theory and the influence charts became a synonym of the pavement mathematical model for several decades (Yang, 1972).

The lack of the pavement system equilibrium was brought up by Burmister (1945). He did not emphasize the early work by Boussinesq (1885) in solving the equations by polynomials but plunged into the development of more refined solutions. But until the publication of the extensive listings of tabulated co efficients by Jones (1962), no significant appreciation of the application of the layered theories was developed by engineers .

The use of the layered systems alone is not sufficient to solve pavement problems. It must be supplemented with the second and third mathematical models in analysing the conditions in pavement components. The subject of mathematical models is divided into two parts:- .

a. the equilibrium of the system; and

b. the displacement and stress in the system components.

(a) Equilibrium of the System: There are two kinds of external forces which may act on bodies. They are surface forces such as hydrostatic pressure and body forces such as gravitational forces, magnetic forces, etc. In studying the equilibrium of bodies under a static loading condition, the surface forces are considered and resolved into stress components parallel to the co-ordinate axes.

(b) Displacement and stress in the system

To compute the displacements and stresses in a pavement structure under loads. The commonly used methods are:

1. Boussinesq's homogeneous linear elastic theory
2. Burmister's linear and non-linear elastic layered theory and
3. The non – Linear finite – element method

The first and second methods have been widely used because of their simplicity. The third method incorporated with non linear soil characterizations is most promising for solving pavement problems provided that the true stress – strain relation of the materials in each layer is known (Ahlvin et al 1974)

The assumptions behind Bo ssinesq elastic theory are:

(i) Material is semi-infinite in extent.

(ii) Material is isotropic

(iii) It is homogeneous

(iv) It is elastic and obeys Hookes' law

Bousinesq equations are theoretically sound for one – layer system – a semi – infinite mass having a distributed load on the boundary surface (Michelow, 1963)

Burinisten (1945) developed solutions for specific two and three layered systems and in doing this assumed the following:

(i) Each layer is composed of material which are isotropic, homogeneous and weightless.

(ii) The system acts as a composite system. That is there is a continuity of stress and /or displacements across the interfaces depending upon the assumptions made regarding the interface conditions and

(iii) Most solutions assume materials that are linearly elastic.

The use of finite element method in layered system offers certain advantages such as statical checks on the results, ease of treating variations in dimensions and physical properties and non-ambiquity in the boundary condition (Newmark, 1949)

2.2.2 Experimental Field Approach

There are probably more than twenty methods which have been developed in this approach. These methods are developed by professioner groups based on their years of experience in design and construction. Some of the methods are california method california Bearing Ration (CBR) method, American Association of State Highway and Transportation official(AASHTO) method, shell method et cetera. Some of these methods are semi – empirical while some are entirely emirical. They are one part of the attempts in finding lasting solutions to the problem of pavement design.

2.2.2.1 Improvement on Elastic Layer Theory

Aside from the pioneering efforts of Boussinesq (1985) Burmister (1945), and others, many researchers have improved on the elastic layer theory, appreciably. Some of the researchers are: Peatie and ullidtz (1982) Smith – Witczak (1981), and Hadley et al (1973).

Peatie and Ullidtze (1982) simplified theory, for multilayered stress – strain analysis was developed for the computation of tensile and com-pressive strains within the pavement. The relevant equations are:

$$\sigma_t = \sigma_z = \sigma_o\left(1 - \frac{1}{[1+(\frac{a}{z})^2]^{3/2}}\right) \qquad (2.01)$$

$$\epsilon_z = \sigma_o(1+\mu)\left(\frac{z/a}{(1+(\frac{a}{z})^2)^{3/2}} - (1-2\mu)\left[\frac{z/a}{(1+(\frac{a}{z})^2)^{1/2}} - 1\right]\right) \qquad (2.02)$$

z

$$\epsilon_t = \frac{1}{E}\,\frac{(1-\mu)}{2\mu}\,(\sigma_z - E\epsilon_z)) \qquad (2.03)$$

where

σ_z = Vertical stress at depth z due to wheel load;

σ_o = Actual wheel load;

ϵ_z = Compressive vertical strain ;

ϵ_t = Tensile tangential strain ;

μ = Poisson's ratio;

E = Modulus of elasticity;

a = Tyre contact radius;

z = Depth of pavement layer.

Smith – witczak (1981) proposed an analytical model for the prediction of single valued Equivalent Base moduli in the horizontal and vertical directions for elastic layered system. These models incorporate the derived material characterization for various pavement component sizes. Some of the equation are as follows:

$$\text{Log } E_{et} = 1.079 - 0.511 \log h_1 - 0.008 \log h_2 - 0.155 \log E_1 + 0.279 \log E_3 + 0.888 \log k_1 \qquad (2.04)$$

$$\text{Log } E_{ev} = 0.959 - 0.430 \log h_1 - 0.073 \log h_2 - 0.122 \log E_1 + 0.294 \log E_3 + 0.848 \log k_1 \qquad (2.05)$$

Where

E_{et} = equivalent base moduli in horizontal direction (psi)

E_{ev} = equivalent base moduli in vertical direction (psi)

h_1, h_2 = Layer thickness for surface and base respectively.

E_1 & E_3 = Modulus of elasticity of surface and subgrade

K_1 = Intercept regression constant from the modulus test plot.

Hadley et al (1973) interative solutions layered theory was also developed to calculate stresses and strains in a pavement. Some of the relevant equations are:

For tensile stress in bottom of layer

$$\begin{aligned}\text{Log}_{10}\, \sigma_2 \times 10^3 = [&1{,}333.84 + 46.868\,(E_2 - 5.5) - 23.774\,(h_1 - 7.5) \\ &-3.361(E_2 - 5.5)(h_2 - 7.5) - 11.008\,(E_2 - 5.5)^2 \\ &+1.6306(E_2-5.5)^3+4.3026(h_1-7.5)(h_2-7.5)-39.310(E_1-5.0) \\ &-21.572(E_4-8.0)+1.7254(E_4-8.0)^2 -80.972(h_1-7.5) \\ &+4.0607(E_2-5.5)(h_1-7.5)+3.4019(h_1-7.5)^2] \quad \ldots\ldots\ldots\ldots \quad (2.06)\end{aligned}$$

For tensile strain in bottom of base layer

$$\begin{aligned}\text{Log}_{10}\, \epsilon_2 \times 10^3 = \sigma_2[&1{,}412.12 - 34.516\,(E_2 - 5.5) - 19.891\,(h_2 - 7.5) \\ &-3.677(E_2 - 5.5)(h_2 - 7.5) + 2.0650\,(E_2 - 5.5)^2 \\ &+3.9519(h_1-7.5)(h_2-7.5) - 37.103(E_1-5.0) \\ &-21.293(E_4-8.0)+1.7920(E_4-8.0)^2 \\ &-76.216(h_1-7.5) + 3.9517\,(E_2-5.5)(h_1-7.5) \\ &+2.7973\,(h_1-7.5)^2] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.07)\end{aligned}$$

where

$E_1, E_2 \ldots E_4$ are as defined earlier

h_1, h_2..... are as defined earlier

ϵ_2 and σ_2 are the strains and stress of the pavement respectively

There is also the BISAR programme equation to calculate the value of young modulus as defined by elastic theory. The important equations are:

$$\sigma_z = \sigma_o \left[1 - \frac{1}{(1+(a/z)^2)^{3/2}}\right] \quad \text{.......(2.08)}$$

$$\sigma_H = \frac{\sigma_z}{2}\left[1+2u) - \frac{2(1+2u)}{[(1+(a/z)^2)^{1/2}} + \frac{1}{(1+(a/z)^2)^{3/2}}\right. \quad \text{.......(2.09)}$$

$$\epsilon_t = \frac{1}{E'} \quad [\sigma_y - \mu \ (\sigma_H + \sigma_z] \quad \text{.......(2.10)}$$

$$\epsilon_z = \frac{1}{E''} \quad [\sigma_z - \mu(\sigma_H + \sigma_y] \quad \text{.......(2.11)}$$

$$\epsilon_z = \frac{1}{E''} \quad [\sigma_z - 2\mu\sigma_H] \quad \text{.......(2.11a)}$$

Where

σ_o	=	as earlier defined
σ_z, σ_H	=	vertical and horizontal stresses respectively
E, ϵ_t, ϵ_z	=	as earlier defined
a, μ, z	=	as earlier defined

$\sigma_H = \sigma_Y$

2.3 LATERITIC SOILS AS ROAD PAVEMENT CONSTRUCTION MATERIALS

2.3.1 Lateritic Soil: According to Alexander and Cady (1962), lateritic soil is a highly weathered material rich in secondary oxides of iron, aluminium or both and it is nearly of bases and primary silicates, but may contain large amounts of quartz and kaolinites. Vallerga et al, (1969) defined lateritic soil as a hardened material formed by the process of primary weathering of residual soils where sesquioxides are present in the material or through the secondary enrichment and cementation of transported or residual soils.

Gidigasu (1976) identified five profiles of lateritic soil formation in which the three main horizons are described as 'laterite zone' - the upper horizon where laterization is usually at its peak and concretionary pieces abound; soil matric is clayey, of stiff to hard consistency and texture generally granular. The next horizon is called 'mottled zone' where laterization is less than in the previous zone and its soil types are generally very silty, or clayey. The third horizon called 'pallid zone' is the lowest section of the completely weathered rock segment. This zone is predominantly silty and very often contains degraded mica flakes. The latter two horizons are collectively referred to as 'saprolites'. (Ajayi, 1960). Figure 2.1 shows the typical lateritic soil profile.

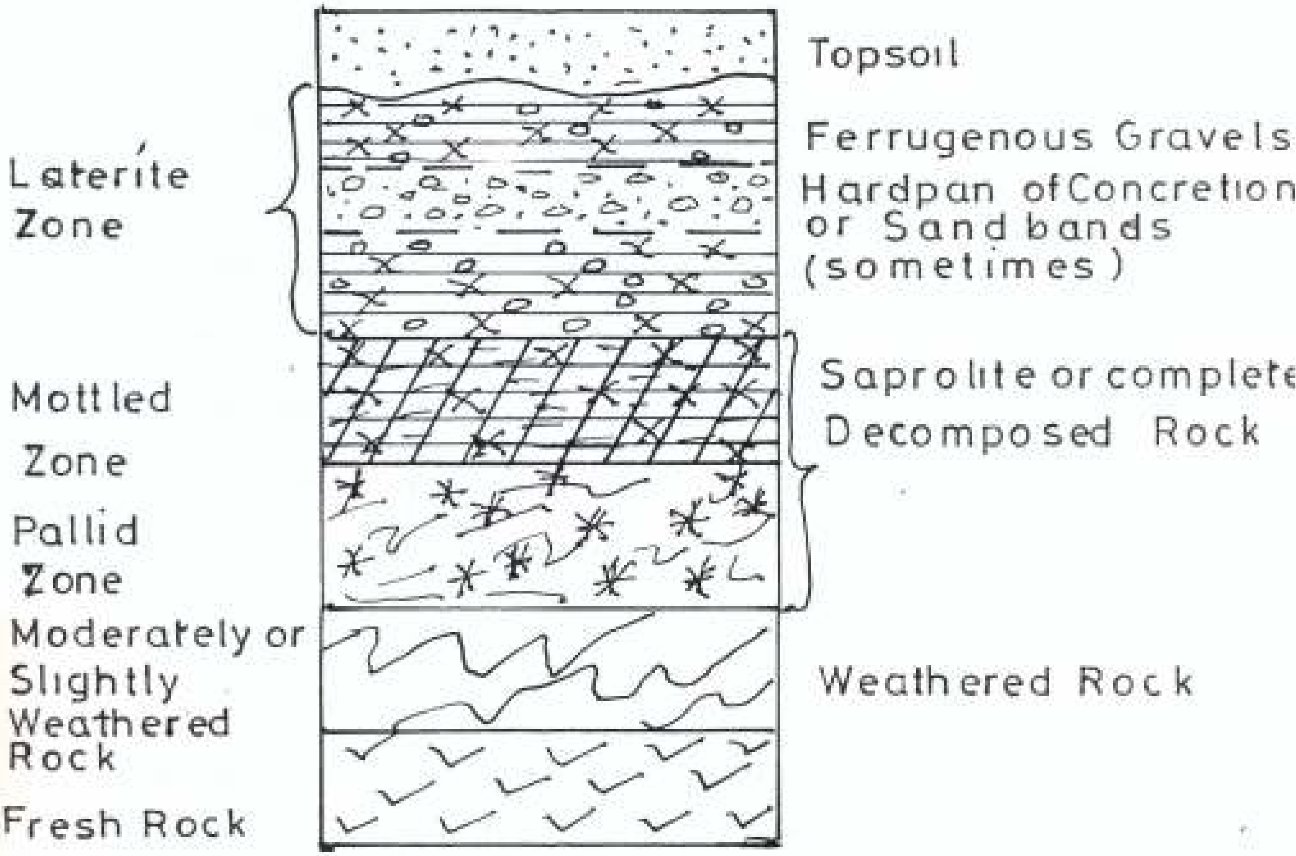

Fig 2-1 Typical Lateritic Soil Profile

Lateritic soil constitutes an important material for engineering uses and a relatively large proportion of the world especially the tropical zone is covered with laterite (Giuseppe, 1969). Motir and Machar (1969) described lateritic soil as the end product of weathering produced from the chemical decompositon of rocks under tropical conditions. Levinson (1974) gave some favourable conditions for the formation of lateritic soil as high temperature, strongly óxidizing environment, long duration of weathering, a chemically unstable rock, high seasonal rainfall and intense leaching.

2.3.2 Specifications for Lateritic Soil for Road Bases in Tropical Countries:

Townsend, et al (1971) carried out a remarkable analysis and comparison of material specifications for roads in use in fourteen African Countries - Nigeria, Angola, Cameroon, Ghana, Ethiopia, Gabon, Gambia, Ivory Coast, Kenya, Mali, Malawi, Senegal, Uganda and Zambia. They concluded that many failures of pavements have resulted from the indiscriminate use of laterites through the adoption of specifications and classification systems developed for temperate climate area soil proper-ties. In particular, they argued that both AASHO and U.S. Corps of Engineers specifications for pavements are overly stringent and inappli-cable to laterite materials. Similar views were expressed by Mitchell and Sitar (1982) and the *UNDP sponsored work of Messrs Scott Wilson Kirkpatrick and Partners of 1970/71 on Northern area roads.

Table 2.1 shows the recommended specification for selection of base course materials (Townsend, et al 1971).

Clare (1960), described Central African laterites as comprising nodules, nearly all about 10mm in diameter. Stewart (1970) found Malaysian laterites to be composed of gravel between 2mm and 25mm in size. Dowling (1966) published grading distributions for high-level and footslope laterites from Nigeria. From the study of these people, Charman (1975) recommended materials selection criteria for lateritic sub-bases and roadbases under thin bituminous surfacings in the tropics. This is shown in Table 2.2.

2.4 SOIL STABILIZATION

Capper and Cassie (1976) defined soil stabilization as any process which increases or maintains the natural strength of the soil. It includes compaction, drainage and the stabilization of banks by sowing of grass and planting of trees. More usually, the term is restricted to the improvement of soil properties by treatment with additives. (Capper and Cassie, 1976). Jackson (1983) described soil stabilization as a term used for the improvement of soils either as they exist in situ or when laid.

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*UNDP - United Nations Development Project

Table 2.1: Specification for Selection of Base Course Materials

	Road Classification			
Criteria	Class I (Heavy Traffic)	Class II (Medium Traffic)	Class III (Low Traffic)	Current FMW Specification
Design CBR	80 or 100 (Min)	60 or 70 (Min)	50 (Min)	80 (Min) 30 (Sub-base)
(LL) x (Fines % Passing 0.075mm)	600 (Max)	900 (Max)	1250 (Max)	1050 (Max) 125 - 375 (for Gravel Base)
Liquid Limit (LL)	30 or 45 (Max)	40 (Max)	--	30 (Max)
Plasticity Index (PI)	10 or 15 (Max)	12 (Max)	--	12 (Max)
(PI) x (Fines % Passing 0.075mm)	350 (Max)	350 - 400 (Max)	500 (Max)	420 (Max) 100 - 300 (for Gravel Base)
Los Angeles Abrasion	65 (Max)	65 (Max)		

Source: Townsend et al. (1971)

Table 2.2: Recommended Material Selection Criteria for Lateritic Gravel Sub-Bases and Road-Bases

Use	Road-Base			Sub-Base
Type of Road	Minor	Major Rural and Urban		All
Traffic ($x10^6$ esa)	< 0.3	0.3 - 1	1 - 3	< 3
Grading	Grading Modulus ≮ 1.5 (1)			
Plasticity Index (%) Moist, wet tripical Seasonal tropical Semi arid & arid	 ≯ 12 ≯ 15 ≯ 20	 ≯ 10 ≯ 12 ≯ 15	 ≯ 6 ≯ 10 ≯ 12	} } ≯ 25 }
Plasticity Modulus Moist, wet tropical Seasonal tropical Semi arid & arid	 ≯ 300 ≯ 400 ≯ 500	 ≯ 200 ≯ 250 ≯ 350	 ≯ 150 ≯ 200 ≯ 250	 ≯ 500 ≯ 750 ≯ 1250
CBR (%)	≮ 45(3)	≮ 65(4)	≮ 80(4)	≮ 25(3)
Durability Los Angeles Abrasion value (%) 10% fines value (saturated) (KN)	 ≯ 65 n.s	 ≯ 50 ≮ 50		 n.s n.s

n.s = not specified

Stabilization is often achieved by injecting a fluid into the pore spaces in soil or by introducing admixtures to the soil (Ingles and Metcalf, 1973).

Many materials have been used as additives to stabilize soils, including chlorines, lignins, molasses and several of the synthetic resins. However, the only materials now used on a large scale are cement, lime and bitumen (Scott, 1974).

2.4.1 Cement Stabilization: This term is applied to various materials in which natural soil washed or processed granular materials, or waste products such as slag or fuel ash, are improved by the admixture of a small proportion of portland cement, generally not more than 10 or 15% (Kezdi, 1979). Most soils including tropical soils and laterites can be stabilized with cement. The additon of cement develops a higher shearing strength and a greater resistance to ingress of water. The high average temperature of the tropics are advantageous for the stabilization of laterite soils with cement, because strength gains are faster (De Graft-Johnson and Bhatia, 1969). Meanwhile, high temperature and delay in compacting cement stabilized soil can cause problems. According to West (1959), a 2-hour delay in compaction of cement stabilized soil reduced the strength by half.

2.4.2 Lime Stabilization: Hydrated lime has been widely and successfully used as stabilizing agent for fine-grained plastic soils. When lime is added to a fine-grained soil, several reactions such as cation exchange and flocculation-agglomeration occur. A lime-soil pozzolanic reaction may occur resulting in the formation of various types of hydrated calcium silicate and aluminium cement agent. The pozzolanic reaction is time and temperature dependent and the reaction may continue for several years (Thompson, 1972). Woods and Yoder (1952) suggest two stages for lime stabilization of soil. First is the alteration in the film of water which surrounds the clay minerals accompanied by a flocculation of clay particles which would end in making the clay soil less plastic. The second stage is a chemical reaction from which a cementious bond develops.

Improvements in soil workability, water resistance and reduction in swell properties, plasticity, shear strength, CBR and stiffiness are effected by the cation exchange and flocculation-agglomeration reactions and the strength, stiffness and durability properties are further improved as a result of the cementing material formed in the lime-soil pozzolanic reaction. (Molenaar, 1993).

2.4.3 Bitumen Stabilization: Bituminous stabilization is of most value in granular soils. The technique employed varies with the prevailing climatic conditions, the choice of major methods depending on how rapidly the prepared soil dries out. The main object of the method is to coat individual grains with bitumen and to ensure that the stabilizer used - emulsion, a cut-back bitumen or a cut-back bitumen mixed with wax or a wetting agent are well incorporated to provide cohesion and waterproofing especially in sandy soil (Capper and Cassie, 1976).

The types of soil generally treated in this way are clean sands or sands containing a small amount of cohesive materials. The bitumen seals the pores of the soil reducing the permeability and may also increase the shear strength considerably by binding the soil particles together (Scott, 1974).

2.4.4 Stabilized Lateritic Soil: Stabilization reduces the sensitivity of lateritic soils to moisutre changes and subsequent loss of strength (Sherwood, 1968). According to George, et al (1992), stabilized lateritic soil is that lateritic soil that had been subjected to every physical, physico-chemical and chemical method to enable them serve better their intended engineering purposes. It is the lateritic soil whose properties had been improved in order to use it as roadway bases, sub-bases and occasionally surface courses in highway engineering (Atkin, 1980).

2.4.5 Specifications for Stabilized Natural Gravels: Specifications for soil stabilization with cement and lime give ranges of soil properties which are appropriate for material selection. Mixtures of the soil with varied amounts of stabilizer are subsequently tested to check strength or durability criteria. Table 2.3 shows the specifications for stabilized natural gravels. British and French guidelines and also Bulman (1972) Stabilization Practice in 11 African Countries are used as reference.

Table 2.3: Specifications for Stabilized Natural Gravels

Specification Source and Reference		TRRL (UK) (1977)		CEBTP (France) (1980)				
Traffic Category Accumulated Traffic (x10^6 esa)		n.s		T3 - T5 >10		T1 - T2 <10		T3 - T5 >10
Type of Material		Soil Stabilized with		Subbase Lateritic Gravel Treated with		Road base Gravel Improved with		Road base Gravel Stabilized with Cement
		Cement	Lime	Cement	Lime	Cement	Lime	
GRADING	Max size (mm)	n.s	n.s	10-30	10-50	10-50	10-50	10 - 50
	Uniformity coef	>5	>5	n.s	n.s	n.s	n.s	>10
	% Passing 0.425mm	n.s	≮15	n.s	≮15	n.s	≮15	n.s
	% Passing 0.080mm	n.s	n.s	<35	<35	<35	<35	<35
PLASTICITY	Plasticity Index (%)	n.s	>10	<30	10-30	<25	10-25	<25
	Plasticity Modulus	n.s	n.s	< 2500	< 2900	< 2000	< 2000	Mix in Place <1500; Plant Mix <700
	Organic Content (%)	n.s	n.s	<1.5	<1.5	n.s	n.s	n.s
CBR TESTING	CBR after 3 days in air & 4 days soaking (%)	<100	n.s	>100	>60	>160	>160	n.s
	CBR after 28 days in curing in air (%)	n.s	>100	n.s	n.s	n.s	n.s	
	Requeired Compaction: *Method *Dry Density (%mdd) *Moisture Content	B.S 1924 Test 13 (1)		B.S 1377 Test 13 95 Optimum				
STRENGTH	Unconfined Compressive Strength 7 days curing in air (N/mm²)	n.s		n.s				1.8 - 3.0
	Unconfined Compressure Strength 3 days in air 4 days soaking (N/mm²)	≮1.7 (2)		n.s				≮0.5

2.5 Effects of Temperature and Moisture on Pavement System

2.5.1 General:

Temperature and moisture are the two most popular environmental factors which could cause the distress and failure of pavements without the application of wheel load. Cracks and disintegrations may occur outside the traffic wheel path on pavements (Yang, 1972). The general mechanism by which the environment influences pavement behaviour and performance are:-

(i) the effect on engineering properties of component materials, such as strength and tractive resistance;

(ii) the effect on the integrity of materials, such as durability and physico-chemical disintegration;

(iii) the effect on volumetric change and the resulting internal stress equilibrium in the pavement system.

Thompson (1970) has prepared a comprehensive report on this subject.

2.5.2 Stress Induced by Temperature:

If a linear thermal expansion exists for a given pavement material (by implication, this assumption does not apply to asphaltic material), the unit linear expansion is Et and the unit volumetric expansion is approximately 3 Et. If the stress-strain property of the material is linearly elastic, the direct axial stress as a result of thermal expansion is given by

where $\sigma = \varepsilon t E$ (2.12)

E = thermal stresses

E = modulus of elasticity of pavement material

t = temperature of the pavement

The thermal stress in a pavement is practically governed by the elastic modulus of the pavement materials (Yang, 1972). The higher the modulus of elasticity, the more pronounced the thermal stress will be. The change of temperature with depth known as the thermal gradient and expressed by (Δt) will result in warping of the pavement.

The radius of curvature (R) of the warping and the resulting internal moment M in the pavement components are:-

$$\frac{1}{R} = \frac{\epsilon t}{z} \qquad (2.13)$$

and
$$M = \frac{EI}{R} \qquad (2.14)$$

For a homogenous pavement component, the bending stress (σ_t) is given by

$$\sigma_t = 1/2\,\epsilon Eh\frac{\Delta t}{\Delta z} \qquad (2.15)$$

Where

h = pavement thickness.

The total temperature stresses consist of the bending stress and the direct axial stress - equations (2.12) and (2.15) respectively. The combined thermal stress becomes

$$\sigma_t = \epsilon E(t_b \pm \frac{1}{2}h\frac{\Delta t}{\Delta z}) \qquad (2.16)$$

E = modulus of elasticity of pavement material

t = temperature of the pavement

The thermal stress in a pavement is practically governed by the elastic modulus of the pavement materials (Yang, 1972). The higher the modulus of elasticity, the more pronounced the thermal stress will be. The change of temperature with depth known as the thermal gradient and expressed by (Δt) will result in warping of the pavement.

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$$\frac{1}{R} = \frac{\in t}{z} \quad \text{.........................} \quad (2.13)$$

and
$$M = \frac{EI}{R} \quad \text{.........................} \quad (2.14)$$

For a homogenous pavement component, the bending stress (σ_t) is given by

$$\sigma_t = 1/2 \in Eh\frac{\Delta t}{\Delta z} \quad \text{.........................} \quad (2.15)$$

Where

h = pavement thickness.

The total temperature stresses consist of the bending stress and the direct axial stress - equations (2.12) and (2.15) respectively. The combined thermal stress becomes

$$\sigma_t = \in E(t_b + \frac{1h\Delta t}{2\Delta z}) \quad \text{.........................} \quad (2.16)$$

where

t_b = temperature range at the bottom of a pavement.

If a linear thermal gradient exists in the pavement system, the zero temperature variation is encountered at a depth (d) below the surface, and the above equation (equation 2.16) can be rewritten for the maximum combined stress as:

$$\sigma_t = \varepsilon E(d - \frac{h}{2}) \frac{\Delta t}{\Delta z} \quad \text{.......................................} \quad (2.17)$$

For d > h

The effect of temperature variation on the bending stress of pavement components is usually not as significant as the effect of wheel load. According to George, et al (1992), the strength of a lime-soil is little affected at temperatures 20°C and 35°C and only had significant improvement at 50°C. The most serious combination of wheel load and thermal stress is encountered during the winter months or cold nights, when the surface termperature is far cooler than the base of the pavement (Yang, 1972).

2.5.3 Moisture Movement in Soil: Moisture held above the water table is retained by surface tension at the point of contact of the particles or in the soil pores and capillaries. The water held above the water table will be in equilibrium when the upward component of the surface tension force is equal to the gravitational force acting on the suspended water. The height to which the water will rise in the capillary tube is related mainly to the surface tension and the radius of the meniscus given by:

$$H = \frac{2T}{\rho r g} \qquad (2.18)$$

where

T = surface tension per unit length of boundary

r = radius of tube or meniscus

ρ = density of water

H = height of capillary rise

In a soil mass, the moisture content is based on the equilibrium of soil suction. Moisture may migrate from areas of low moisture content, such as sand layers, to areas of higher moisutre content, such as clay and silt layers. The distribution of moisutre is erratic and random, and there is no moisture gradient responsible for moisutre movement. Meanwhile, the temperature gradient in a soil can also affect the migration of moisture in addition to the soil suction.

The supporting capacity of any pavement material is governed by the relation -

$$S = C + (p - u) \tan \varnothing \qquad (2.19)$$

Where

S = shearing strength of pavement materials

C = cohesive bond

P = normal pressure

U = excessive pore water pressure

$\varnothing$= internal frictional angle

An excessive increase of moisture in the pavement will greatly reduce the bearing capacity of the pavement and would consequently increase its deflection (Yang, 1972).

2.6 FLEXIBLE PAVEMENT DESIGN

2.6.1 General: According to Ahlvin (1962), pavement design objective is to provide acceptable pavement structure that can withstand the deteriorating effects of traffic and environment for the service life of the facility. Hejal et al (1971), described a properly designed, constructed and maintained pavement as a major factor in providing economical, efficient, safe, convenient and comfortable highway travel. To achieve the objectives stated above, different parameters are considered in pavement design.

2.6.2 Pavement Design Variables: The parameters considered in the design of flexible pavement are traffic loading, climate or environment, material characteristics, design period, cost of construction and maintenance (Wright and Paquette, 1987). All these could also be called elements of thickness design.

The thickness design of pavements requires a large number of complex factors to be considered. The factors are:-

According to Yoder and Witczak (1976), all pavement design procedures can be grouped into two basic approaches. The first approach is based on a fixed traffic level for failure inherent, but not obvious. This procedure uses a relationship established between pavement thickness and a single-wheel load for various subgrade support values. The second approach relies on a fixed standard vehicle type and it expresses pavement thickness as a function of number of repetitions to failure of the standard vehicle for various subgrade supports.

For the procedure in the first approach, there is need to equate the design multiple-wheel gear into an equivalent single-wheel load (ESWL). Normally, the critical or "design" vehicle is used in this approach. The standard vehicle approach involves the determination of the equivalent damage effects of all vehicle types in the mix expressed in terms of the number of repetition of the standard vehicle. The factors that relate the relative amount of damage of a particular vehicle to that caused by the standard vehicle are denoted as equivalent wheel load factors (EWLF).

In general, pavement design objectives attempt -

1. the development of design strategy of material economy, safety and serviceability in recognition of the variational nature of design factors;
2. the maximization of the accuracy of prediction of serviceability, safety and physical deterioration of alternatives being considered.
3. the minimization of cost of materials - cost and benefit analysis;
4. the maximum use of local materials and labour in design strategies;
5. the minimization of cost of design - studies and material testing, etc.
6. the consideration of design alternatives;

All the above (points 1 to 6) are considered within the design constraints of funds available, time available, minimum level of performance before rehabilitation, materials available, minimum or maximum layer thickness, minimum time between overlays, material testing capabilities, capability of the design model available,

quality, extent and type of information available (Adeyeri, 1996).

2.7 CIVIL ENGINEERING SYSTEMS ANALYSIS

2.7.1 General: According to Templeman (1982), civil engineering systems analysis studies the practice of civil engineering as a creative, decision-making process for which a systematic approach and a knowlege of some efficient decision-making methods are invaluable. It provides a logical, comprehensive framework for the study of civil engineering decision-making. Ramo (1969) described civil engineering systems as an intellectual discipline for mobilizing science and technology to attack complex, large-scale problems in an objective, logical, complete and thoroughly professional way. In attempting to solve engineering problems, civil engineering systems analysis considers the problems or project under system, system design and optimization.

2.7.2 System: A system may be considered to be an ordered assembly of interconnected elements that transforms, in a given time reference, certain measurable inputs into measurable outputs (Ossenbruggen, 1994). It is an assembly of methods, procedures or techniques united by regulated interaction to form an organised whole.

2.7.3 System Design: A system design of a project comprises of a rational and orderly series of steps that lead to the best decision for a given set of conditions (Merritt, 1976).

2.7.4 Optimization: According to Rao (1979), optimization is the act of obtaining the best result under given circumstances. It is the process of finding the conditions that give the maximum or minimum value of a function. The optimum seeking methods are also known as mathematical programming techniques and are generally studied as a part of operations research. The mathematical programming techniques are useful in finding the minimum of several variables under a prescribed set of constraints (Rao, 1979).

Linear programming, non-linear programming, dynamics programming, geometric, quadratic and integer programming are some of the methods used in mathematical programming techniques. In this project, linear programming will be considered and will thus be emphasized.

2.7.5 Linear Programming: Rao (1979) defines linear programming as an optimization method applicable for the solution of problems in which the objective function and the constraints appear as linear functions of the decision variables. The constraint equations in a linear programming problem may be in the form of equalities or inequalities.

The standard form of linear programming can be expressed as -

Minimize $f(X_1, X_2 \ldots X_n) = C_1, X_1 + C_2X_2 + \ldots C_nX_n$ ...... (2.20)

Subject to the constraints

$a_{11}, X_1 + a_{12}, X_2 + \ldots a_{1n}X_n = b_1$ .... (2.12)

$a_{21}, X_1 + a_{22}, X_2 + \ldots a_{2n}X_n = b_2$ .... (2.12)

$a_{m1}, X_1 + a_{m2}, X_2 + \ldots a_{mn}X_n = b_n$ .... (2.23)

$X_1, X_{21} \ldots X_n \geq O$ .... (2.24)

where

cj, bj and a_{ij} =		known constants
X_j	=	decision variables
i	=	1, 2, ... m
and j	=	1, 2, ... n

These equations written in compact form reduce to

$$\text{Minimize } f(X_1, X_2, \ldots X_n) = \sum_{j=1}^{n} C_j, X_j \quad \ldots\ldots\ldots \quad (2.25)$$

Subject to the constraints

$$\sum_{j=1}^{n} a_{ij}X_j = b_i, \quad \ldots\ldots\ldots \quad (2.26)$$

$$= 1,2, \ldots m$$

and

$$X_j \geq O \quad \ldots\ldots\ldots \quad (2.27)$$

$$j = 1,2 \ldots, n$$

The characteristics of the linear programming problem can be summarised as follows:-

1. The objective function can be expressed in minimization or maximization type;
2. All the constraints are of either equality or inequality type;
3. All the decision variables are non-negative.

To change from minimization to maximization or vice-versa, the following transformation would occur to the standard form:-

(a) The maximization of a function $f(X_1, X_2 ... X_n)$ is equivalent to the Minimization of the negative of the same function, that is,

$$f = C_1,X_1 + C_2X_2 + ... C_nX_n \text{ is equivalent to}$$

Maximization $f = -f = -C_1,X_1 - C_2X_2 + ... -C_nX_n$

This approach is vice-versa.

(b) In most of the engineering optimization problems, the decision variables represent physical dimensions and hence the variables X_j have to be non-negative.

(c) If a constraint appears in the form of a "less than" type of equality as

$$a_{k1}X_1 + a_{k2}X_2 + ... a_{kn}X_n + X_n +_1 < b_k$$

Similarly, if the constraint is in the form of a "greater than" type of inequality as

$$a_{k1}X_1 + a_{k2}X_2 + ... + a_{kn}X_n > b_k$$

it can be converted into the equality form by substracting a variable as

$$a_{k1}X_1 + a_{k2}X_2 + ... + a_{kn}X_n - X_n +_1 = b_k$$

where x_{n+1} is a non-negative variable known as surplus variable.

In linear programming problems, there are m equations in n decision variables. If $m > n$, there would be $m - n$ redundant equations which could be eliminated. If $m = n$, the constraints are inconsistent and the equation has no solution. The case $m < n$ corresponds to an undetermined set of linear equations which, if they have one solution, have an infinite number of solutions.

The aim of linear programming is to find one of these solutions satisfying equations (2.21 to 2.24) and yielding the minimum or maximum of the objective function.

Arora (1989) suggested an optimum design process as shown in Figure 2.2

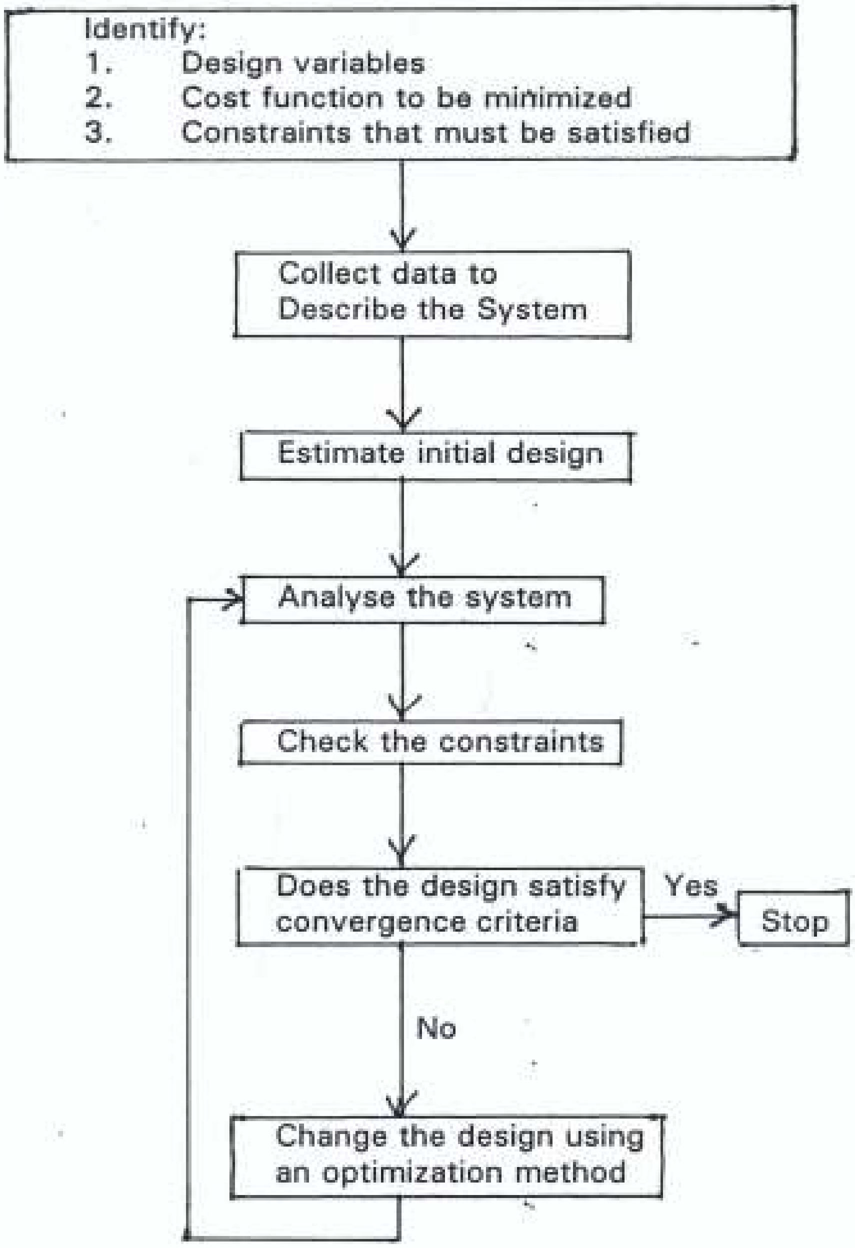


Fig. 2.2: Optimum design process flow-chart.

2.8 FLEXIBLE PAVEMENT DESIGN FRAMEWORK

2.8.1 Design Framework: Design framework for flexible pavement design consists of a step by step approach to evaluating the elements of the design as composed in the framework. Essentially, the major elements of this framework could be handled in four steps given thus:

1. Traffic analysis;
2. Design traffic and tyre contact pressure
3. Structural analysis; and
4. Optimal pavement component analysis.

1. **Traffic Analysis:** In the Nigerian Highway design practice, this is basically the determination of the anticipated traffic for the design life of the pavement and it is expressed as the number of vehicles per day exceeding 3 tonnes (30kN) loaded weight (Highway Manual, Part 1, 1973). Lane distribution of these vehicles must be taken into consideration for multi-lane roadways. Here, also the traffic growth rate is considered. To estimate traffic growth rate, use is generally made of a growth factor which is determined from a statistical study of the traffic data. According to Yoder and Witczak (1976), traffic at any time in the future is given by

$$(EAL)_n = \frac{(EAL)_o\,(1+r)^n\,(365)}{Log_e(1+i)} \quad \text{......................... (2.28a)}$$

where

$(EAL)_n$ = EAL at any year n

$(EAL)_o$ = initial daily EAL on the day the road is opened to traffic

r = rate of traffic increase or traffic growth rate expressed as a per cent per year.

EAL = Equivalent Axle Load.

Meanwhile, EAL_o is given by

$$EAL_o = \frac{TESA}{100\ Trucks} (\%Truck) \frac{(ADT)}{2} \quad \text{......................... (2.28b)}$$

Where

TESA = Total Equivalent Single Axle
ADT = Average Daily Traffic.

2. **Design Traffic and Tyre Contact Pressure:** According to Otte et al (1982), the design traffic and tyre contact pressure are dependent on material type and could be expressed as three different numbers of load repetitions as shown in Table 2.4. To obtain these parameters, it is recommended that the estimated repetitions, n, of each axle load group, including overloads be estimated for the design period. For the cement-treated layer, the number of heavy wheel loads expected on the pavement is the critical design parameter to control the rate and amount of crack development.

   An average tyre contact pressure of 520KPa will be assumed for this research.

Table 2.4 Definition of Design Traffic

Material Type	Design Traffic
Bituminous Cement-Treated	Total number of axle repetitions T, (e.g. $5x10^6$) numbe (e.g.$5x10^3$) of heavy wheel loads (e.g. 40KN) preferably the full traffic spectrum
Untreated and Subgrade	Number of equivalent 80KN axles (E80) (e.g. 10^6)

Source: Otte et al (1982)

3. **Structural Analysis:** This is mainly the evaluation of subgrade, sub-base, base and surfacing materials in terms of mechanical strength, stiffness modulus and Poisson's ratio. With these evaluated parameters, the practical combinations of materials and layer thicknesses are selected with due consideration of the effects of environment on these materials.

By using any of the layered elastic theories or any other relevant theories, the maximum developed stresses and strains in any of the pavement components could be calculated and compared with the safe working stress and strain of these components.

Paetie and Ullidtz (1982) simplified theory for multi-layered stress-strain analysis is one of the relevant equations for the computation of the tensile and compressive strains within the pavement. The equations are as follows:

$$\sigma_z = \sigma_o\left(1 - \frac{1}{[1+(\frac{a}{z})^2]^{3/2}}\right) \quad \ldots\ldots\ldots\ldots \quad (2.29)$$

$$\epsilon_z = \sigma_0(1+\mu)\left(\frac{z/a}{\left(1+\left(\frac{a}{z}\right)^2\right)^{3/2}} - \frac{(1-2\mu)\,z/a}{\left[\left(1+\left(\frac{a}{z}\right)^2\right)^{1/2} - 1\right]}\right) \ldots \qquad (2.30)$$

$$\epsilon_t = \frac{1}{E}\,\frac{(1-\mu)}{2\mu}\left(\sigma_z - E\epsilon_z\right) \ldots\ldots \qquad (2.31)$$

where

σ_z = Vertical stress at depth z due to wheel load;

σ_0 = Actual wheel load;

ϵ_z = Compressive vertical strain on top of the subgrage layer;

ϵ_t = Tensile tangential strain at the bottom of bituminous layer;

μ = Poisson's ratio;

E = Modulus of elasticity;

a = Tyre contact radius;

z = Depth of pavement layer.

Otte (1978) stated that the allowable tensile strain ϵ of a cement-treated material depends on the material's strain at break ϵb, and the number of load repetitions, N_f. That is,

$$\frac{\epsilon}{\epsilon_b} = 1 - 0.11 \log N_f \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.32)$$

Or

$$N_f = 10^{9.1\left(1-\frac{\epsilon}{\epsilon_b}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.33)$$

Where

N_f = number of load repetitions

ϵ = allowable tensile strain of a cement-treated material.

ϵ_b = material strain at break.

4. **Optimal Pavement Component Analysis:** This is where civil engineering system analysis is relevant. Having evaluated the desired parameters of the pavement components, an optimal pavement component analysis would follow which will aid the selection of the most efficient practical combinations of materials and layer thickness of the pavement with due consideration of all constraints.

CHAPTER THREE

MATERIALS AND METHODS

3.1 METHODOLOGY

The methodology uses a systematic approach starting from traffic analysis through traffic loading and pavement response to the selection of the most efficient and cost effective pavement layout.

3.1.1 Traffic Analysis: The traffic census data collected on Akure-Owo Expressway were analysed. The result of the analysis using traffic growth rate calculated from the traffic census fluctuates unreasonably and therefore the growth rate of 7% was adopted as specified in Road Research Laboratory (LR.279). A fixed standard vehicle approach, in which equivalent wheel load factor (EWLF) relates equivalent damage effects of all vehicle types in the traffic mix expressed in terms of the number of repetitions of the standard vehicle, was used for the analysis. The traffic census data were processed to give the average daily traffic (ADT), (estimated by taking the average of vehicles for seven consecutive days), the percentage of passenger cars, trucks, two axle load vehicles and trailers for 1987, that year being the most recent year and the one having the highest traffic census data. A pavement design life of twenty (20) years was adopted for the analysis. Using all these, the equivalent axle load (EAL) for the future (20 years) was estimated.

The design traffic being dependent on material type was expressed as three different numbers of load repetitions on bituminous materials, treated materials and untreated and subgrade materials. For the treated layer, the number of heavy wheel loads expected on the pavement was made the critical design parameter to control the amount and rate of crack development. The total number of axle repetitions (T), irrespective of load, is the required traffic parameter for design of thin bituminous surfacings. Traffic loading on the untreated layers and subgrade was expressed in terms of the number of equivalent 80 KN axle (E80).

An average tyre contact pressur of 520kpa (0.52mpa) was used for the calculations.

3.1.2 Materials Characterization: The fundamental properties of lateritic soils, both stabilized and unstabilized were sourced from laboratory test results on laterite borrow pit materials conducted by Olorunfemi (1984). The laboratory test results on laterite borrow pit materials carried out by Jimoh (1987) were also considered. The lateritic soils were categorized under base, sub-base and subgrade materials. The stabilization of the sub-base materials to upgrade them, as base materials was considered. The additives used were cement and lime. The effects on the strength, compaction and Atterberg Limits of the sub-base material when different percentages of cement and lime were added were considered.

The fundamental properties of bituminous and asphaltic materials such as modulus of elasticity (E) and Poisson's ratio (μ) were obtained from (Singh and Akoto 1981) in Journal of American Society of Civil Engineers (ASCE), Volume 170, No. TE6. The modulus of elasticity of the stabilized laterite base especially that of the soil-cement was estimated from an equation developed by Otte, (1978) relating the unconfined compressive strength to the average modulus of elasticity (E_b) of cement-treated materials. The calculation is shown in Appendix II by equation 6.01. The representative stiffness or modulus of elasticity of the subgrade was estimated from the approximate relationship suggested by Heukelom and Klomp (1962). The equation and the stiffness estimation are also shown in Appendix II equation 6.02. The Poisson's ratio adopted for the asphalt, stabilized laterite, unstabilized laterite were 0.4, 0.35 and 0.4 respectively. Based on Mitchell (1976), the equation relating the tensile strength of a soil-cement to its unconfined compressive strength is as stated in Appendix II. Using the equation, the tensile strength of the soil-cement was estimated. For the resilient modulus of untreated and treated lateritic soils, the equation relating the resilient modulus (M_R) of the material to the axial deviator stress (σ_d), the recoverable axial strain (ϵ_a), the bulk stress (σ) and the regression constants (K_1 and K_2) from the $\log M_R$ versus $\log\sigma$ plot was considered. The $\log M_R$ versus $\log\sigma$ was plotted to evaluate the regression constants (K_1 and K_2) of the base material.

As for the lime stabilized lateritic soil, the factors influencing its resilient properties were considered. The cyclic data on soil-lime and soil-cement obtained from Jimoh (1987) were used to plot a graph of resilient modulus as a function of applied stress (deviator stress).

3.1.3 Safe Working Stresses and Strains of Pavement Materials: The safe working tensile strains at the bottom of bituminous and asphaltic concrete materials were considered. Also considered were the effects of mix stiffness in bituminous material and void content in asphaltic concrete on the strains of these materials.

For the treated materials especially the soil-cement, the allowable tensile strains were computed using equations 6.04 and 6.05 in Appendix II. The safe working compressive strain at the subgrade surface was adopted to control pavement rutting. The standard of judgement used by Dorman and Metcalf (1965) was adopted for this study.

3.1.4 Environmental Effects: The effects of the climate on the pavement was simulated by using soaked CBR values of the sub-base and subgrade materials especially at the optimal selection of the pavement layer thicknesses. Some of the parameters used in the American Association of State Highway and Transportation Officials (AASHTO) road test were adapted in a bid to correlate theory and practice. The parameters considered were regional factor, soil support, layer co-efficient and the serviceability index. The stresses and strains induced in the pavement by temperature were not considered.

3.2 PRELIMINARY COMBINATION OF PAVEMENT MATERIALS, LAYER THICKNESS, LOADING AND PAVEMENT RESPONSE

3.2.1 <u>Preliminary Combination of Pavement and Layer Thicknesses</u>:

Different combinations of the treated and untreated lateritic materials with four different layer thicknesses each of the surfacing and stabilized base were evaluated on one hand. And on the other hand, four different layer thicknesses each of the surfacing and treated sub-base were also evaluated. In each case, one layer thickness of each of the sub-base and base was considered. For the material combination, the base course material were considered stabilized with untreated sub-base course materials. Later, the sub-base course material was considered treated while the base-course material was untreated.

3.2.2 <u>Loading and Pavement Response</u>: The load from the design traffic and the resulting tyre contact pressure were used to calculate the developed pavement stresses and strains at critical points of the pavement in connection with the preliminary combination of pavement material and layer thicknesses. The developed values of these stresses and strains in each of the arrangements stated above were compared with the safe working values for the materials.

Any of these arrangements whose developed stresses and strains were less than the values for safe working stresses and strains of the material were chosen as preliminary practical combination of materials and layer thicknesses.

The tensile stress of the soil-cement was evaluated by multiplying the tensile strength calculated by the relationship based on Mitchell (1976) with the fatique strength ratio of the material obtained from Hadley et al (1973).

As a result of the failure criteria chosen, the stresses and strains at the following critical areas of the pavement were evaluated:

1. For no-cracking criterion, the tensile stresses and strains occuring at the bottom of the stabilized layer and the induced tensile strain at the bottom of asphalt layer were considered;
2. For cracking criterion, the tensile strains at the bottom of the asphalt layer and the compressive strain at the top of the subgrade were considered;

In relation to the failure criteria used, the following approaches were used in selecting the preliminary thickness of the stabilized bases. First, the pavement was assumed to possess slab-type behaviour and thus the layers were assumed to be infinite in longitudinal and transverse directions. For this, base thickness selection was made on the basis that the induced stresses and strains are not greater than the fatigue tensile strength of the stabilized layer, the tensile strains in both the asphalt and the stabilized layers and the vertical compressive strain at the top of the subgrade. Secondly, it was assumed that the treated bases and sub-bases will crack shortly after construction. For this two cases were considered. These are:-

(a) cracks are closely spaced and the governing parameters here are stated in cracking criterion in (2) above;

(b) cracks that are wide would lead to slabs becoming finite and edge-loading was assumed instead of interior loading. In this case, the induced stresses would be higher than those calculated assuming an infinite slab.

A cracked stabilized layer was considered as behaving like granular materials. Due to cracking, the horizontal modulus of the layer will be highly reduced and the stabilized layer becomes anisotropic instead of isotropic which works with elastic theory. To overcome this complication, a cracked treated layer was assumed as an equivalent, untreated isotropic material with elastic modulus of 750Mpa and 1100 Mpa which fall between 500 and 1500 Mpa as suggested by Otte et al (1982). In addition to this, the factors suggested by Wilson and Pretorius (1970) was applied to accommodate local increases in the vertical strain immediately beneath the cracks in treated layers. The critical areas that were considered for the no-cracking criterion is as specified in (1) above.

3.3 DESIGN EQUATIONS:

Three equations from elastic layered theory were considered. The equations are:-

i. Peatie and Ullidtz (1982) simplified theory for multi-layered stress-strain analysis;

ii. Hadley et al (1973) equations on iterative solutions of layered theory;

iii. The BISAR Programme equations for estimating the value of Modulus of Elasticity as defined by elastic layered theory;

Out of these equations, it was only the - BISAR Progamme equations that gave consistent results of the stresses and strains at the specified points in the pavement. These equations were therefore adopted for the stress and strain analysis of the pavement using Smith-Witczak (1981) single valued equivalent base moduli. The equations are listed thus

For BISAR Programme:

$$\sigma_z = \sigma_o \left[1 - \frac{1}{(1+(a/z)^2)^{3/2}} - \right]$$

$$\sigma_H = \frac{\sigma_z}{2}\left[1+2\mu\right) - \frac{2(1+2u)}{[(1+(a/z)^2)^{1/2}} + \frac{1}{(1+(a/z)^2)^{3/2}}$$

$$\epsilon_t = \frac{1}{E'} \left[\sigma_y - \mu \ (\sigma_H + \sigma_z\right]$$

$$\epsilon_t = \frac{1}{E''} \left[0_z - 2\mu\sigma_H\right]$$

For Smith – Witczak (1981) equation

$$\text{Log } E' = 1.079 - 0.511 \log h_1 - 0.008 \log h_2 - 0.155 \log E_1 + 0.279 \log E_3 + 0.888 \log k_1$$

$$\text{Log } E'' = 0.959 - 0.430 \log h_1 - 0.073 \log h_2 - 0.122 \log E_1 + 0.294 \log E_3 + 0.848 \log k_1$$

Where

σ_o = Actual wheel load

σ_z, σ_H = Vertical and horizontal stress respectively

ϵ_t, ϵ_z = Tensile and compressive strains respectively

μ = Poissons ratio

E', E'' = Single valued equivalent base modulus for tensile and compressive strains respectively.

h_1, h_2 = Layer thickness for surface and base respectively.

E_1 & E_3 = Modulus of elasticity of surface and subgrade

K_1 = Intercept regression constant from the modulus test plot.

$\sigma_H = \sigma_Y$

A computer programme was run on these equations with different pavement preliminary material combinations and layer thicknesses which resulted into an output of over 200 different values of stresses and strains.

3.4 OPTIMAL MATERIAL COMBINATIONS AND LAYER THICKNESSES

The stresses and strains from computer output were compared with the maximum permissible stresses and strains at the critical points of the pavement to select those that are less than the safe working stresses and strains at those points. Three or four from the pavement layouts immediately following those that satisfied these conditions were selected as preliminary layouts.

The final and optimal layout of the pavement was selected by using modified Hejal et al (1971) minimization equation for selection of flexible pavement components by applying civil engineering systems analysis method. The Hejal et al (1971) equation after being modified using relevant parameters is given by:

Minimize $S = 5.257h_1 + 1.183h_2 + 0.864h_3$

Subject to the following constraints

$$a_1 h_1 + a_2h_2 + a_3h_3 \geq \overline{SN}$$

$$h_1 + h_2 + h_3 \geq H_{min}$$

$$h_1 \geq 1''$$

$$h_2 \geq 8''$$

$$h_3 \geq 8''$$

$$h_1 \leq 4''$$

$$h_2 \leq 20''$$

$$h_3 \leq 8''$$

$h_1, h_2, h_3 \geq 0$

where

S = total cost of pavement in dollars per square foot

h_1, h_2, h_3 = thickness for surface base and sub-base respectively

a_1, a_2, a_3 = layer coefficients for surface base and sub-base respectively.

$\overline{SN}$ = weighted structural number of the pavement

H_{min} = Minimum total pavement thickness

The layer thicknesses of the preliminary layouts were used as lower and upper thickness constraints. Some of the conditions and parameters specified by the American Association of State Highway and Transportation Officials (AASHTO) design guide were adapted to suit the tropical environment. Flexible pavement design nomographs from AASHTO design guide were used to estimate the weighted structural number (SN) of the pavement. Finally, modified Hejal et al (1971) equation was solved to produce the optimal pavement layout. The constraints equations were tested to make sure the conditions specified were fulfilled. Modified Hejal et al (1971) equations and their solutions are in Appendix II under item 6.17

The optimal pavement layout was then compared with other existing layouts from two former design procedures for the tropical environment.

3.5 SOURCES OF DATA:

The data used for this study were obtained from various national and international publications in engineering and other related subjects. The data and their sources are itemized thus:-

3.5.1 Traffic Data: The data on traffic census on Akure-Owo Expressway were sourced from Ondo State Ministry of Works, Transport, Lands and Housing, Akure. Other data such as traffic growth rate and design traffic were obtained from Road Research Laboratory (LR.279), Journal of Transportation Engineering - Transport Nigeria, 1981 and Otte et al (1982) in Journal of American Society of Civil Engineers (ASCE), Volume 108, No. TE4, respectively. All these are shown in Tables A-01(a) to A-07(b) in Appendix 1

3.5.2 Fundamental Material Properties Data:

i. **Stabilized and Unstabilized Lateritic Soil -** Data on material properties were obtained from laboratory tests on stabilized and unstabilized lateritic soils in Nigeria. The additives considered were cement and lime. The cyclic test data for the untreated and the treated laterite base materials were obtained from Jimoh (1987).

ii. **Bituminous Materials -** the data on the fundamental properties of this material were obtained from Ondo State Asphalt Company (OSAC), Akure and from (Otte et al, 1982) in the Journal of American Society of Civil Engineers (ASCE), Volume 108, No. TE4

iii. **Subgrade Materials -** For this study, a laterite soil of A-2-7 class was assumed as the subgrade material. The information on the fundamental material properties were sourced in the same manner as (i) above.

3.5.3 Pavement Failure Criteria Data: Two pavement failure criteria were considered. They are the no-cracking and the cracking criteria. The data for the maximum permissible strain values were obtained from (Otte et al, 1982) in Journal of American Society of Civil Engineers (ASCE), Volume 108, No. TE4

3.6 CHECKING OPTIMAL LAYOUT AGAINST THE DESIGN LIFE:

Aside from comparing the proposed procedure with the former procedures, the optimal layout of soil-cement for a cracking criterion was used to check whether the twenty-year (20) design life earlier used will be enough for the optimal layout or not. The analysis is shown in Appendix II under item 6.25 and the discussion is under item 4.9

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 TRAFFIC ANALYSIS RESULTS:

The traffic analysis results for Akure-Owo Expressway for 1987 is presented in Table 4.01. The analysis shows the axle load distribution and the equivalent (80KN) single axle computations for the year. From the table, it is seen that the total equivalent (80KN) single axle (TESA) per 100 trucks on the road is 35.02 out of which tandem axle is 4.30 (about 12%). The implication of this is that heavy vehicles (trucks) do not ply the road with the same frequency as light vehicles such as private cars and taxi cabs. The table shows that vehicles in the range of 18 to 27KN (1.8 - 2.7 tonnes) have the highest frequency on the road. These light vehicles do not cause as much pavement distress as heavy vehicles.

Table 4.01(a) classifies the traffic along the road. Passenger cars take the "lion's share of 88% while trucks take 12%. . Of the 12% for trucks, trailers take 2% while two axle load vehicles take the remaining 10%. This classification confirms the low frequency of heavy vehicles on Akure - Owo Expressway.

The plate also shows equivalent axle load (EAL) computations using 1987 traffic census data with a design period of 20 years. The traffic growth of 7% was used. The computations show that from the 1987 equivalent axle load (EAL_o) of about 9, the equivalent axle load (EAL_{2o}) twenty years after will be $1.362x10^5$. These figures (9 and $1.362x10^5$) imply that all vehicular

Table 4.01: Axle Load Distribution and Total Equivalent (80KN) single axle Computations for 1987 Traffic Census along Akure-Owo Express Road

Axle Load (KN)	Single Axles per 100 Trucks			Tandem Axle per 100 Trucks		
	Number (No.)	Factor (F)	No. x F	Number (No.)	Factor (F)	No. x F
Under 9	67.5	0.0002	0.0135			
9 - 18	--	0.002	--			
18 - 27	435.5	0.01	4.3550			
27 - 36	88.8	0.03	2.6640			
36 - 45	33.1	0.08	2.6480			
45 - 54	23.7	0.18	4.2660			
54 - 63	21.3	0.35	7.4550			
63 - 72	7.9	0.61	4.8190	0.28	0.03	0.0084
72 - 81	4.5	1.00	4.5000	0.48	0.05	0.0240
81 - 90				1.90	0.08	0.1520
90 - 99				1.84	0.12	0.2208
99 - 108				--	0.17	--
108 - 117				0.83	0.25	0.2075
117 - 126				--	0.35	--
126 - 135				--	0.48	--
135 - 144				--	0.64	--
144 - 153				0.42	0.84	0.3528
153 - 162				0.97	1.08	1.0476
162 - 171				--	1.38	--
171 - 180				0.69	1.72	1.1868
180 - 189				--	2.13	--
189 - 198				0.42	2.62	1.1004
Total			30.7205			4.3003

Total equivalent 80KN single axle (TESA) per 100 trucks on Akure-Owo Express Road = 30.72 + 4.30 = 35.02

Table 4.01(a) Equivalent Axle Load Computation for 1987 Traffic Census along Akure-Owo Express Road (p = 2.5, SN = 4)

Design period = 20 years
*Traffic growth rate = 7%

Initial Daily EAL_o

$$\sum EAL_o = \frac{35.02}{100}(0.12)\frac{(ADTm)}{2}$$

$$= 0.021\ (ADTm)$$

Future Daily EAL_n

$$\sum EAL_n = \frac{EAL_o(365)}{Log_e(1+i)}[(1+i)^n - 1]$$

Substituting values

$$\sum EAL_{20} = \frac{0.021(ADTm)(365)}{Log_e(1.07)}[(1.07)^{20} - 1]$$

$$= 325.10(ADTm)$$

For 1987 ADTm = 419

$$EAL_{20} = 325.10(419) = 1.362 \times 10^5$$

Classification of traffic on Akure-Owo Road (1987)

Persenger cars	=	88%
Trucks	=	12%
Two axle	=	10%
Trailers	=	2%

ADTm = mean of ADT for pickup van, lorries, buses and trailers
* From RRL (LR 279)

loads on Akure - Owo Expressway in 1987 and those in the year 2007, all other things being equal, were and will be 9 and 1.362×10^5 repetitions of the standard axle load (E80). As a result of this, the vertical compressive strain caused by any load on the pavement within the design life must not exceed the one caused by 1.362×10^5 repetitions of the standard load (E80). The compressive strain (ϵ_c) caused by 1.362×10^5 repetitions of (E80) in the subgrade is the maximum permissible vertical compressive strain for this design. Figure 4.01 shows the variation of traffic along Akure - Owo Expressway from 1982 to 1987. The plot is the average daily traffic (ADT) of all the modes of traffic plying the road against the years. The figure also confirms the road as a light vehicle "trafficked" road with cars and taxis having the highest ADT over the years. Cars and taxis are followed by pickup vans which are also light vehicles. It is observed that the traffic census data for 1987 has the highest mean average daily traffic (ADTm) for all the modes and that is why the year was picked for analysis aside from the fact that it is the most recent of all the data collected on traffic census.

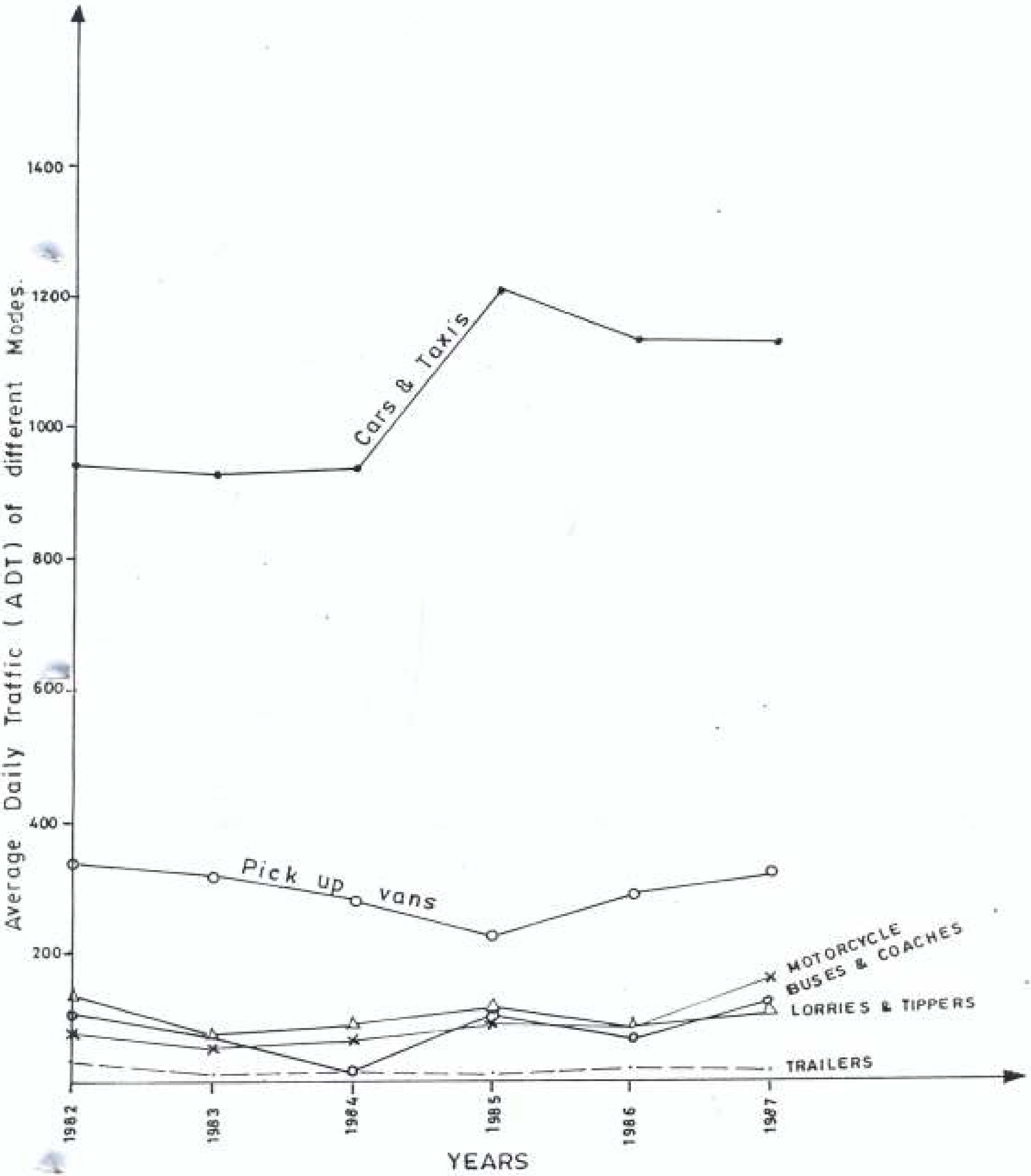


Fig. 4·01 Variation of Traffic with year along Akure – Owo Express Road.

4.2 MATERIALS CHARACTERIZATION:

The natural geo-technical properties of the lateritic soils used are shown in Table 4.02. It is seen from the table that the pavement materials are grouped as base material (A), sub-base material (B) and the subgrade material (C). The table shows materials (B) and (C) as A-2-7 soil while material (A) is an A-2-4 soil meaning that (A) is the best out of the three pavement materials. It has a CBR of 92%. Materials B and C satisfy some of the specifications for materials for road base in the tropics. Because of this, the summary of results on the stabilization of material B is presented in Tables 4.03 and 4.04.

Table 4.03 shows the percentage cement added to the lateritic soil material (B) as ranging from 2 to 8 while table 4.04 shows the percentage of lime added to the same soil using the same range (2% to 8%). From Table 4.03, the CBR of the soil increased from 56% at 2% cement to 300% at 6% cement and dropped to 128% at 8% cement. The table shows the maximum dry density (MDD) of the soil decreasing from 1869KN/M³ at 2% cement to 1585KN/M³ at 8% cement with the optimum moisture content increasing from 15% at 2% cement to 18% at 8% cement.

Table 4.02 Natural Geotechnical Properties of the Lateritic Soils

No.	Material	GEOTECHNICAL PROPERTIES							
		L.L (%)	P.L (%)	P.I (%)	Gs	CBR (%)	Classi-fication	% Clay L.L	% Clay P.I
A	Base	22	15	7	2.67	92	A-2-4	110	35
B	Subbase	44	26	18	2.70	29	A-2-7	264	108
C	Subgrade	44	29	15	2.42	20	A-2-7	308	105

Table 4.03: Summary of Results on Soil-cement

Cement Content (%)	RESULTS						
	CBR (%)	MDD (KN/m³)	OMC (%)	LL (%)	PL (%)	PI (%)	UCS (N/mm²)
0	-	-	-	-	-	-	-
2	56	1869	15	43	29	14	0.711
4	163	1727	19	41	30	11	0.749
6	300	1685	18	41	31	10	0.973
8	128	1585	18	40.5	33	7.5	1.160

Table 4.04: Summary of Results on Soil-lime

Lime Content (%)	RESULTS						
	CBR (%)	MDD (KN/m³)	OMC (%)	LL (%)	PL (%)	PI (%)	UCS (N/mm²)
0	-	-	-	-	-	-	-
2	59	1870	15	41.5	31	10.5	1.123
4	78	1780	19	39	33	6	1.384
6	78	1700	22	36	35	1	1.385
8	45	1650	24	34	38	-	1.122

Similar trends, as shown in table 4.03, also occur in table 4.04. Initially, the addition of lime (2%) increased the CBR of the soil even more than that of cement (2%), but subsequent additions do not show appreciable increase in the CBR. Similarly, lime content inhibits the effectiveness of the compactive effort, though to a slightly less degree. The unconfined compressive strength of soil-lime increases more appreciably from 1.123N/mm² at 2% to 1.385N/mm² at 6% lime before dropping to 1.122N/mm² at 8% lime. From the results of unconfined compressive strength tests, it can be seen that lime impacts more shear strength on the lateritic soil than cement especially at the lower contents of the additives.

The resilient properties of soil-cement and soil-lime where the soils are laterites are influenced by such factors as stress intensity, cement or lime content, moisture content, age of curing and the number of load repetitions, (Boniface and Obi 1972) and (Singh and Akoto 1981).

Tables 4.05 to 4.07 show the cyclic test data on laterite, soil-lime and soil-cement. The same results are shown in Figure 4.02 which shows the relationship between the applied stress (deviator stress) and the resilient modulus of each of the soils. The figure shows that soil-cement has the highest resilient modulus from the three soil samples. This is followed by soil-lime especially at the lower applied stresses. At the higher stresses, lateritic soil has higher resilient modulus than soil-lime. At an applied stress of 280kpa, laterite and soil-lime have the same resilient modulus, the value of which is 240×10^3kpa. The figure also shows that, for all the soils, the resilient modulus decreases as the applied stress increases. Figure 4.03 shows the graph of resilient modulus

Table 4.05: Cyclic Test Data for Lateritic Soil

Cell Pressure	Deviator Stress (Kpa) σ_d	Recoverable Strain (Micron) ϵ_a	Resilient Modulous (MR) (Kpa)	Bulk Stress (Kpa) θ
0	85.5	205	416,529	85.5
40	128.3	381	336,545	248.3
80	248.3	845	294,005	488.0
180	408.3	1576	259,084	918.3

Table 4.06: Cyclic Test Data for Soil-Lime

Cell Pressure	σ_d	ϵ_a	MR	θ
0	80.3	127	632,780	80.3
40	128.3	312	412,243	248
130	388.2	1454	266,976	778
170	488.0	2009	242,871	998

Table 4.07: Cyclic Test Data for Soil-Cement

Cell Pressure	σ_d	ϵ_a	MR	θ
0	84.2	39	2,134,525	84
50	134.7	125	1,076,923	285
100	243.4	324	750,602	543
130	359.9	574	626,885	749
170	522.5	998	523,602	1033

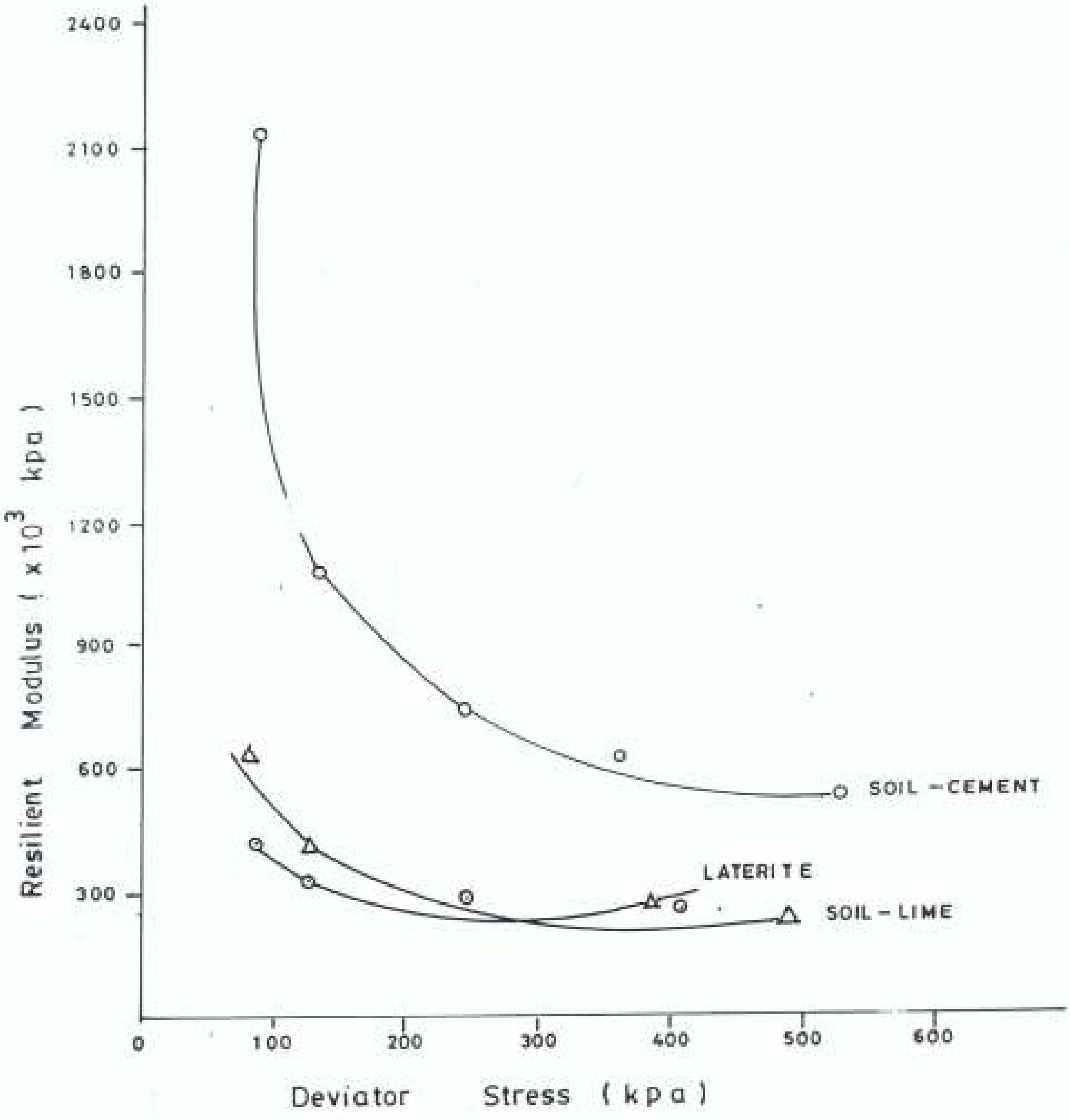


Fig. 4·02 Resilient Modulus as a function of applied stresses.

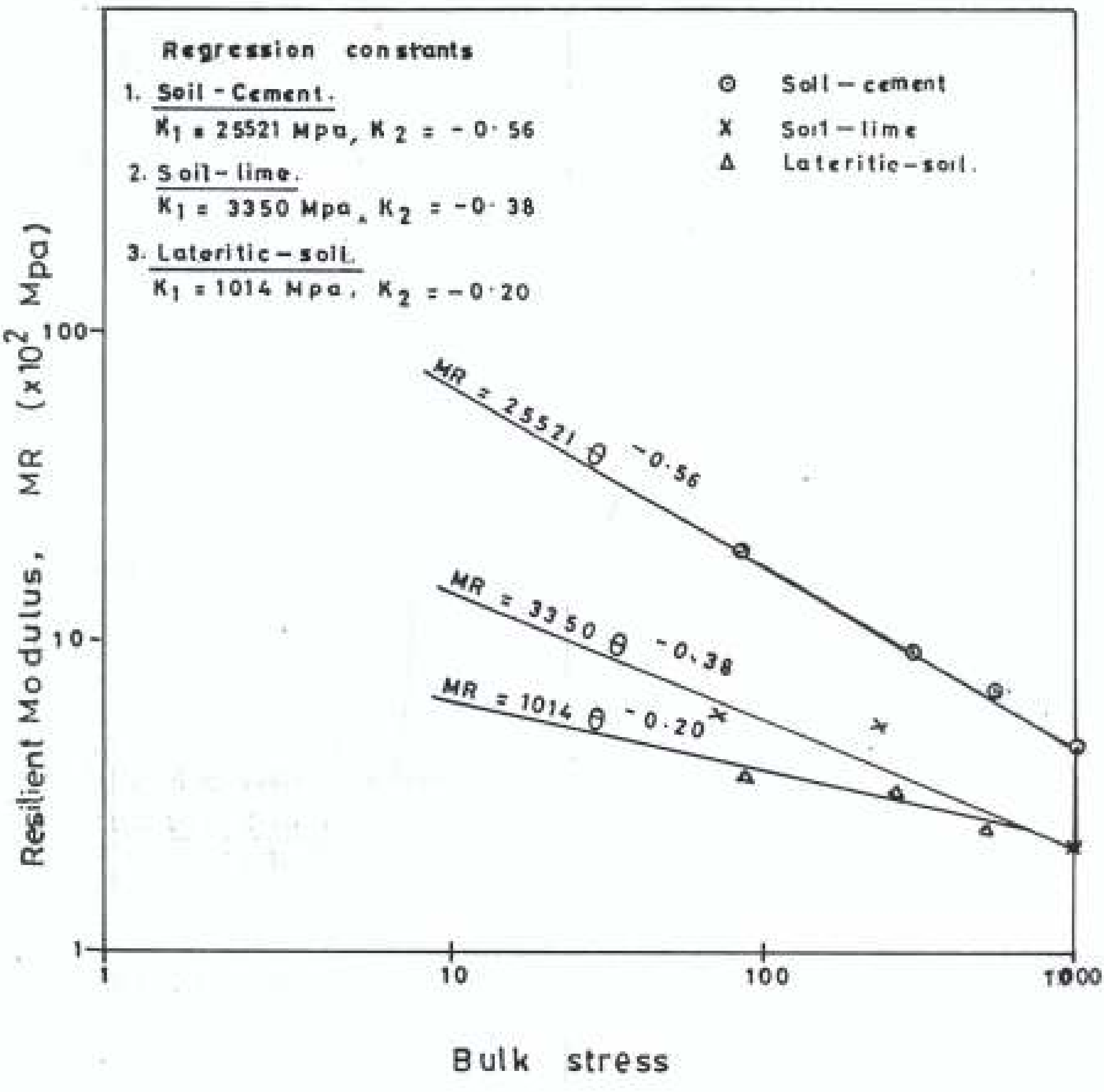


Fig. 4·03 Resilient Modulus versus Bulk stress for Lateritic, Cement and Lime soils.

Source: Jimoh (1987)

versus the bulk stress. It is seen from the figure that the resilient modulus of all the soils decreases as the bulk stress increases. From the figure, the regression constants (K_1 and K_2) were calculated for all the soils and the values are as shown on the figure.

The fundamental properties of bituminous and asphaltic materials, treated and untreated lateritic soils are based on previous studies. The stiffness (E) characteristics of bituminous mixtures depend on temperature and rate of loading. As suggested by Otte et al (1982), one loading rate corresponding to a vehicle speed of 80 - 100km/hr is adopted. A temperature range of between 0°C and 40°C is also adopted. Using all these, a stiffness of 5000Mpa and a Poisson's ratio of 0.4 are used for bituminous and asphaltic materials. For the treated lateritic soils, the modulus of elasticity used ranged between 500Mpa and 4,500Mpa. Based on previous research, the Poisson's ratio used for both of them is 0.35. The relationship established by Mitchell (1976) between the unconfined compressive strength and the average stiffness of a soil-cement is used to generate the stiffness of the soil-cement used here as shown on Table 4.08. Another relationship by the same author is used to estimate the tensile strength of the soil-cement used for this study as it is also shown on table 4.08. From the table, it is observed that increasing the cement content does not give appreciable increase on both the average modulus of elasticity and tensile strength of the soil-cement. This however is a carry-over effect from the value of the unconfined compressive strength of the soil-cement.

Table 4.08: Summary of the Parameters Calculated for Soil-cement

Cement Content (%)	(1) UCS (Mpa)	(2) Average Stiffness (E_s) (Mpa)	(3) Tensile Strength (Mpa)	(4) Fatigue Strength Ratio	(5 = 3 x 4) Tensile Stress (Mpa)
2	0.711	1102	10.18	0.35	3.653
4	0.749	1102	10.20	0.35	3.570
6	0.973	1103	10.30	0.35	3.605
8	0.160	1104	10.38	0.35	3.633

SAFE WORKING STRESSES AND STRAINS OF PAVEMENT MATERIALS:

Table 4.09 shows the maximum permissible strain values for failure analysis. It shows the standard load repetitions (T) and the corresponding permissible tensile strains for asphaltic concrete materials with 5000Mpa stiffness. It also shows the number of equivalent repetitions (E80) and the permissible vertical compressive strain in the subgrade soil corresponding to each of these loads.

For the treated materials especially the soil-cement, the allowable tensile strains (ϵ) depend on the strain at break (ϵ_b) of the material and the number of load repetitions (N_f). Otte (1978) developed a relationship between these parameters. Equations 6.04 and 6.05 in Appendix II define the relationship. Figure 4.04 shows the relationship of elastic modulus and strain at break (ϵ_b) for a soil-cement. In case of a preliminary design, design values of strain at break (ϵ_b) could be obtained from the figure. It is seen from the figure that the strain at break(ϵ_b) decreases as the modulus of the soil-cement increases. This implies that soil-cement with high modulus of elasticity will have a low value of strain at break. Strain at break is that strain value of soil-cement at which the materials fails under the applied load.

Table 4.10 shows the design traffic for different types of pavement materials excluding soil-lime. Limiting values of stresses and strains of materials including soil-lime are shown in Table 4.11. Using both tables with the aid of equations 6.04 and 6.05, the allowable tensile and

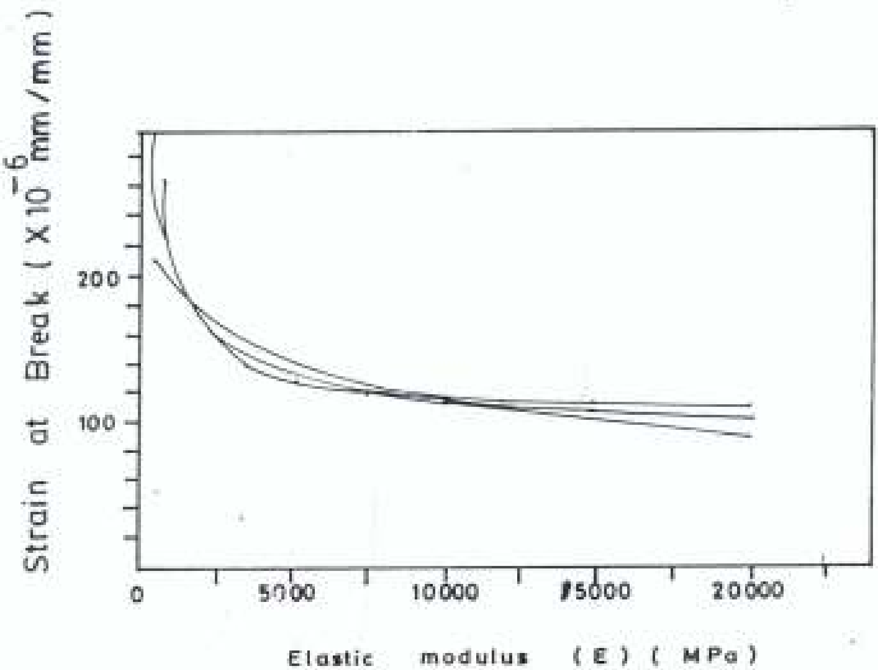


Fig. 4·04. Relationship of Elastic Modulus & Strain at Break for Cement-Treated Gravel.

Source. Otte et al (1982)

Table 4.09: Maximum Permissible Strain Values for Failure Analysis

Number of Standard Load Repetitions (T)	Permissible Tensile Strain (ϵ) $x10^{-6}$mm/mm (E_1 =5000Mpa 9% Void in mix)	Number of Equivalent Repetitions (E80)	Permissible Vertical Compressive Strain in Subgrade Soil (ϵ $x10^{-6}$mm/mm)
10^3	- 880	1 x 10^5	1000
10^4	- 580	3 x 10^5	830
10^5	- 360	1 x 10^6	650
10^6	- 240	3 x 10^6	570
10^7	- 150	1 x 10^7	460
10^8	- 100	> 10^7	400

Table 4.10: Design Traffic for Different types of Pavement Materials

Material Type	Design Traffic
Bituminous materials	Total number of axle repetitions, T. (e.g 5 x 10^6)
Soil-cement	Number of heavy wheel load (e.g 5000 of 40KN)
Untreated and subgrade	Number of equivalent 80KN axles (E 80) (e.g 10^6)

Source: Otte et al (1982)

Table 4.11: Limiting Values of Stresses and Strains for Pavement Materials including soil-lime

Pavement Layer	Load Repetition (T)	σ_{ti} (Mpa)	Limiting ϵ_t	Stresses and Strain (ϵ $x10^{-6}$mm/mm) ϵ_c
Surfacing	10^8		58	260
Base	10^6	0.1 (Weak) 0.3 (Strong)	140	650
Subgrade	10^5		230	1050

Source: Dorman and Metcalf (1965)

Table 4.12: Summary of Allowable Tensile and Compressive Strains

Material	Strains (ϵ $x10^{-6}$mm/mm) ϵ_t	ϵ_c
Asphaltic concrete surfacing	200	
Soil-cement base or sub-base E = 500, 750, 1100, 3000, 4500Mpa	154, 119, 95, 89, 82	
Soil-lime base or sub-base	140	
Subgrade		969

compressive strains for the load repetitions for this study are as shown in Table 4.12. The safe working strains for the asphaltic concrete and subgrade materials are obtained by interpolating for the values - 5,000,000 repetitions and 1.362×10^{6} E80 for surfacing and subgrade respectively. The value (1362×10^{5} Es80)for the subgrade is the one estimated for Akure - Owo Expressway traffic census for a design life of 20 years.

From table 4.12, it is observed that the allowable tensile strain of soil-cement on the average is less than that of soil-lime. It is therefore reasonable to deduce that soil-cement is more vulnerable to distress than soil-lime when used in pavement. The implication of this is that soil-lime can carry more load than soil-cement. This assertion is supported by tables 4.10 and 4.11 where the number of load repetitions "carriable" by soil-cement and soil-lime are 5×10^{3} and 10^{6} respectively. The asphaltic material as observed on the table has the highest allowable tensile strain because it is the most ductile material amongst the three. This is why it can carry as much as 5×10^{6} axle repetitions (see table 4.09).

4.4 ENVIRONMENTAL EFFECTS

The effects of the environment considered were enumerated via the AASHTO design guide with a view to correlating theory with an established internationally accepted road test. Table 4.13 shows the

Table 4.13(a): Estimated Values from AASHTO Design Guide as Adapted to Nigerian Environment for Treated Base

Pavement Layer	CBR (%)	Soil Support Values S-C	Soil Support Values S-L	Weighted Structural Number (SN) S-C	Weighted Structural Number (SN) S-L	Layer Coefficient (a_i) S-C	Layer Coefficient (a_i) S-L
Subgrade	5(Soaked)	3.6	3.6	4.10	4.10	--	--
Sub-base	5, 10(Soaked)	4.8	4.8	3.50	3.50	0.06	0.07(a_3)
Base	163, 78(Unsoaked)	9	8.5	1.9	2.10	0.14	0.27(a_2)
Surfacing	--	--	--	--	--		0.44(a_1)

Conditions

Present Serviceability Index (PSI) = 2.5

Regional factor (subgrade soaked) = 2.5

Modulus of surfacing (E_1) = 7.26 x 10^5psi (5000Mpa)

Daily equivalent 18-kip single axle load applications for 20 years = 1.362 x 10^5 (E80)

S-C = Soil-cement
S-L = Soil-lime

Table 4.13(b): Estimated Values from AASHTO Design Guide as Adapted to Nigerian Environment for Treated Sub-base

Pavement Layer	CBR (%)	Soil Support Valuers S-C	Soil Support Valuers S-L	Weighted Structural Number (SN) S-C	Weighted Structural Number (SN) S-L	Layer Coefficient (a_i) S-C	Layer Coefficient (a_i) S-L
Subgrade	5(Soaked)	3.6	3.6	4.10	4.10	--	--
Sub-base	163, 78(Unsoaked)	9	8.5	1.90	2.10	0.14	0.27(a_3)
Base	78(Unsoaked)	8.5	8.5	2.10	2.10		0.11(a_2)
Surfacing	--	--	--	--	--		0.44(a_1)

Conditions

Present Serviceability Index (PSI) = 2.5

Regional factor (subgrade soaked) = 2.5

Modulus of surfacing (E_1) = 7.26 x 10^5psi (5000Mpa)

Daily equivalent 18-kip single axle load applications for 20 years = 1.362 x 10^5 (E80) estimated values from AASHTO nomographs as adapted to Nigerian environment with the conditions stated under the table. From the table, the soil support values for the soaked subgrade and sub-base materials are 3.6 and 4.8 respectively; while those of the unsoaked soil-cement and soil-lime base materials are 9 and 8,5 at CBR of 163% and 78% respectively. The weighted structural number (SN) for both subgrade and sub-base for the two soils are 4.10 and 3.50 respectively. The weighted structural numbers for the bases of soil-cement and soil-lime are 1.9 and 2.10. These values imply that soil-lime is structurally stronger than soil-cement. The layer co-efficients shown for the sub-base are for soaked CBR 5% and 10% and the one for the base are for Moduli 1100 and 3000Mpa respectively. The layer co-efficient for surfacing is for a modulus of 5000Mpa.

Although, some of the values may be conservative, they however, give reasonable approximations that can be used in practice. The nomographs for estimating the values on table 4.13 are in Appendix II and are shown as figures 6.01 and 6.02

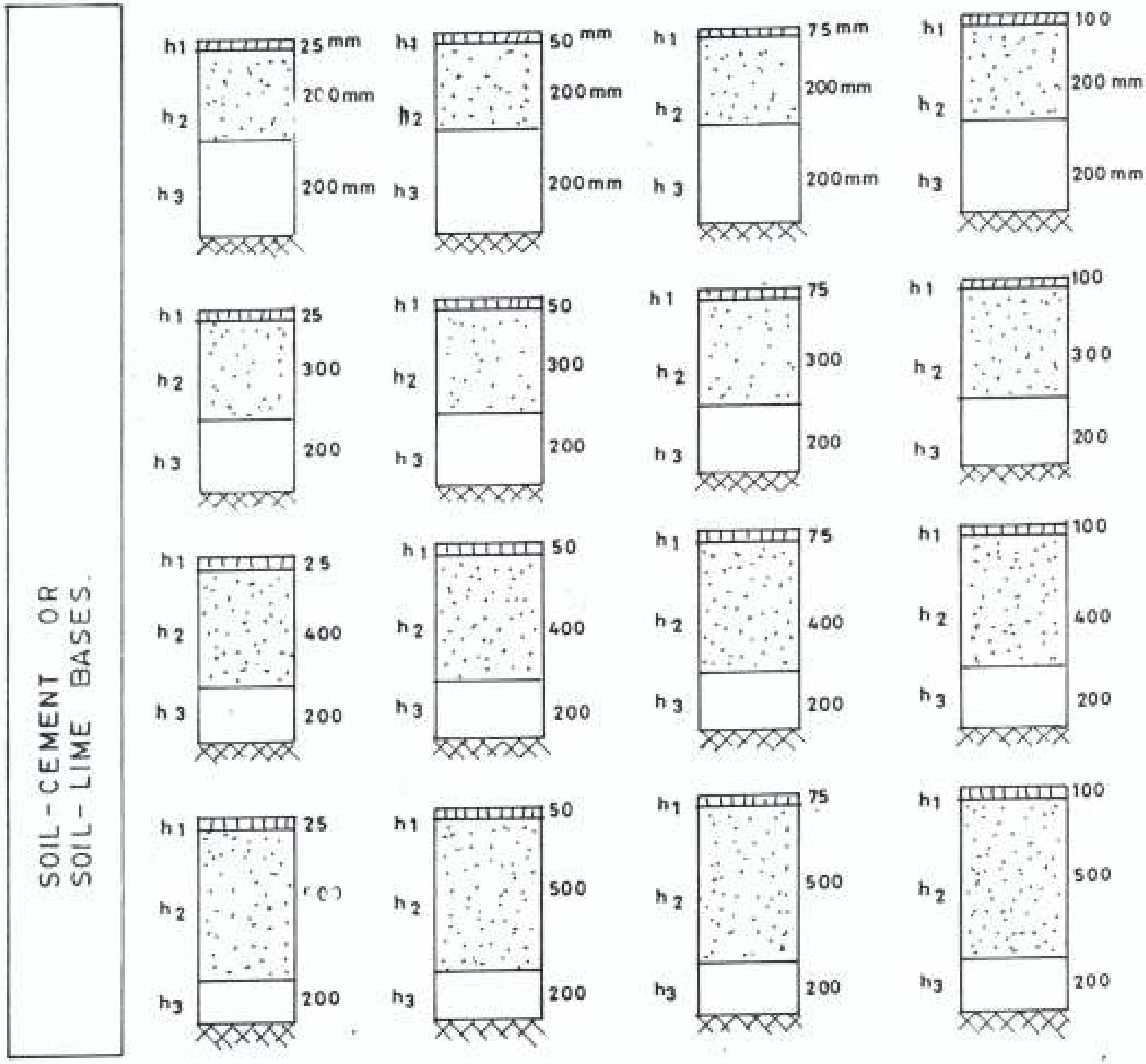
4.5 RESULTS OF PRELIMINARY COMBINATION OF PAVEMENT MATERIALS AND LAYER THICKNESSES

Figures 4.05 and 4.06 show the preliminary layouts of pavement materials with treated base and with treated sub-base respectively. For figures 4.05, the thicknesses of surfacing and treated bases vary from 25mm to 100mm for the surfacing and from 200mm to 500mm for the base. For all these, the laterite sub-base thickness is held constant at 200mm. For figure 4.06, the base thickness is held constant while surfacing and sub-base thicknesses are allowed to vary. These layouts are loaded for the stresses and strains to be calculated at critical points. This will be constituted into what is termed the pavement response.

4.6 LOADING AND PAVEMENT RESPONSE

Figure 4.07 shows a typical pavement loading arrangement for this study. The loading on the pavement is 40kN. The tyre contact pressure and radius are 520kpa (0.52Mpa) and 156mm respectively. The thickness of the surfacing is represented by h_1. Those of base and sub-base are represented by h_2 and h_3 respectively. The moduli of elasticity and the Poisson's ratio of the surfacing, base, sub-base and subgrade are symbolized as E_1; μ_1; E_2; μ_2; E_3; μ_3; and E_4; μ_4 respectively.

Tables 4.14 and 4.15 show the input parameters for the pavement structural analysis.Table 4.14 shows the material combination as asphaltic surfacing, soil-cement or soil-lime basecourse and laterite sub-base. Table 4.15 shows asphaltic surfacing, laterite base and soil-cement or soil-lime sub-basecourse. The responses of these arrangements in terms of stresses and



KEY.

h_1 Bituminous surfacing

h_2 Soil-cement or Soil-lime bases

h_3 Laterite soil Sub-base

Sub grade

Fig. 4·05 Preliminary Layouts of Pavement Materials with treated Bases.

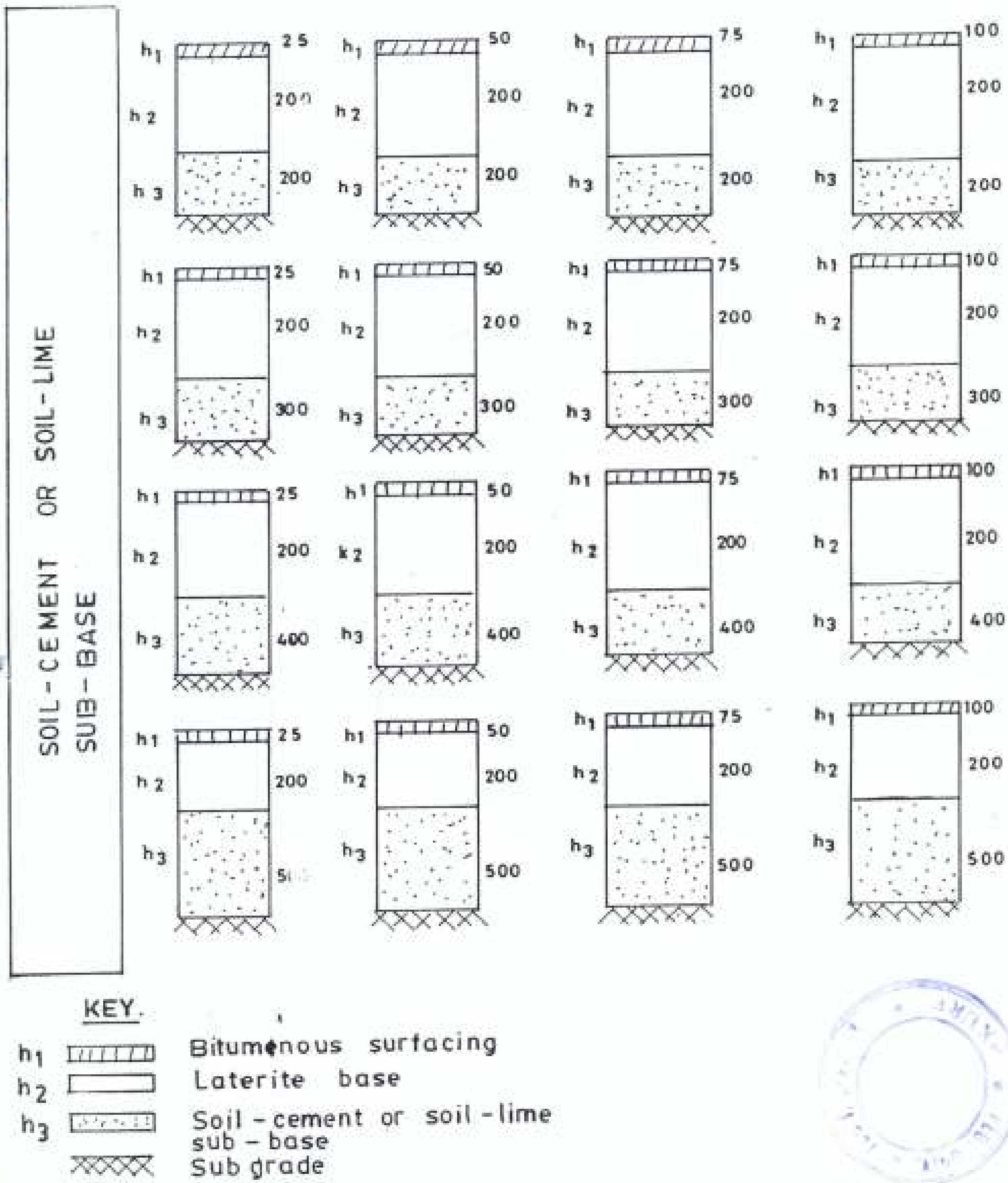


Fig. 4·06 Preliminary Layout of Pavement materials with treated sub-base.

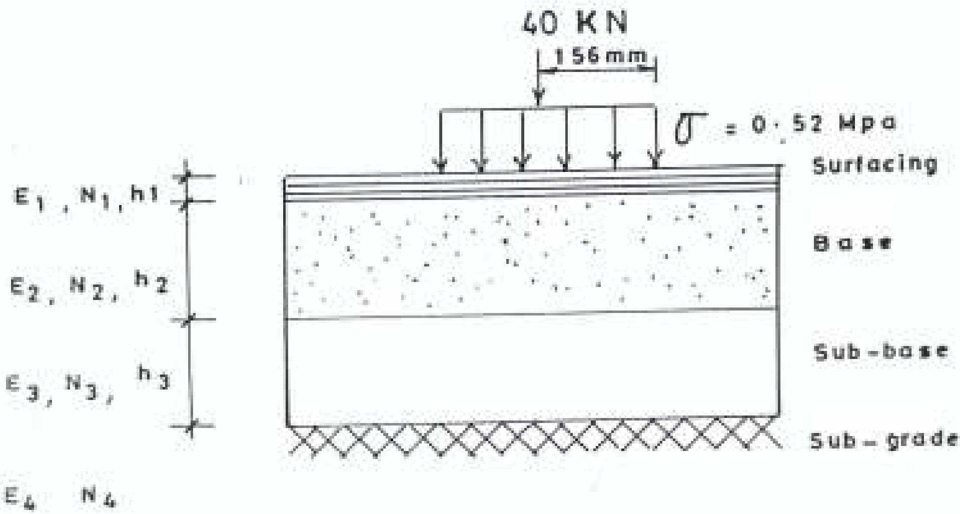


Fig. 4.07 Typical Pavement loading Arrangement.

Table 4.14: Input Parameters for the Design Analysis with respect to Treated Base Course

Parameters	Values Used	Computation
Asphaltic Concrete Layer		
Thickness h_1	25, 50, 75, 100mm	4
Modulus E_1	5000Mpa	
Poisson's Ratio μ_1	0.40	
Soil-cement and Soil-lime Base Layer		
Thickness h_2	200, 300, 400, 500mm	8
Modulus E_2	500, 750, 1,100, 3,000, 4,500Mpa	10
Poisson's Ratio μ_2	0.35	
Laterite Sub-base Layer		
Thickness h_3	200mm	1
Modulus E_3	250Mpa	1
Poisson's Ratio μ_3	0.40	
Subgrade Layer		
Modulus E_4 = 10CBR	50, 100, 200Mpa	3
Piosson's Ratio μ_4	0.40	
Tyre Load	40KN	1
Tyre contact pressure	520Kpa	

Table 4.15: Input Parameters for the Design Analysis with respect to Treated Sub-base

Parameters	Values Used	Computation
Asphaltic Concrete Layer		
Thickness h_1	25, 50, 75, 100mm	4
Modulus E_1	5000Mpa	
Poisson's Ratio μ_1	0.40	
Laterite Base Layer		
Thickness h_2	200mm	1
Modulus E_2	250Mpa	1
Poisson's Ratio μ_2	0.40	
Soil-cement and Soil-lime Sub-base Layer		8
Thickness h_3	200, 300, 400, 500mm	10
Modulus E_3	500, 750, 1,100, 3,000, 4,500Mpa	
Poisson's Ratio μ_3	0.35	
Subgrade Layer		
Modulus E_4	50, 100, 200Mpa	3
Piosson's Ratio μ_4	0.40	
Tyre Load	40KN	1
Tyre contact pressure	520Kpa	

strains at the critical areas of the pavement are shown in Tables 4.16 to 4.20 for soil-cement. It is observed from Table 4.16 that as the thickness of the asphaltic layer increases, the tensile strain value also increases. This is because as the thickness of the asphalt increases, the ductility reduces and thus the fragility increases. The implication of this is that thicker asphaltic layer after a point offers little or no advantage over an optimal thickness.

Tables 4.17 to 4.20 show the stresses and strains at the bottom of soil-cement basecourse for different moduli. The general pattern of the tables and as against the pattern in asphalt surfacing is that, the tensile (negative) and compressive (positive) strains reduce or decrease as the depth increases. It is also observed that higher modulus gives rise to smaller strains below the soil-cement basecourse. The compressive vertical strain on top of the subgrade also shows this pattern. Tables 4.21 to 4.22 show the stresses and strains at the bottom of soil-lime basecourse. The pattern is the same as for those in tables 4.17 to 4.20. The responses of the pavement are shown by Tables 4.23 to 4.26 when the sub-base is treated. All these tables show decreasing values of the stresses and strains as the depth of pavement increases.

Figures 4.08 and 4.09 show the effects of base thickness and sub-base thickness on the tensile strains at the bottom of stabilized bases and sub-bases respectively. It is observed from the figures that as the modulus, base and sub-base thicknesses increase, the tensile strains at their bottom reduce. This implies that the tensile strain is a function of the stiffness and thickness of the treated layers. The depth of the treated layers too affect the strains at the bottom

Table 4.16: Calculated Stresses and Strains at the bottom of Asphalt Layer for Soil-cement and Soil-lime

Soil-Cement			Strains ($^{-6}$ x10mm/mm		Soil-Lime	
Depth (Z) (mm)	Stresses (N/mm^2)				Strains ($^{-6}$x10mm/mm)	
	σ_Z	σ_H	ϵ_t	ϵ_z	ϵ_t	ϵ_z
25	0.518	0.314	4	53	4	53
50	0.503	0.175	19	73	19	73
75	0.474	0.066	30	84	30	84
100	0.431	0.006	35	87	35	87

Table 4.17: Calculated Stresses and Strains at the bottom of Cement Stabilized Base Layer

Soil-Cement: E_2 = 1100Mpa			Strains ($^{-6}$x10mm/mm)		E_2 = 500Mpa Strains ($^{-6}$x10mm/mm)	
Depth (Z) (mm)	Stresses (N/mm^2)					
	σ_Z	σ_H	ϵ_t	ϵ_z	ϵ_t	ϵ_z
225	0.231	0.062	-106	239	-234	526
250	0.202	0.058	-94	210	-208	463
275	0.178	0.053	-84	185	-184	408
300	0.157	0.048	-74	164	-164	361
325	0.139	0.044	-66	145	-146	320
350	0.124	0.040	-59	130	-130	285
375	0.111	0.036	-53	116	-117	255
400	0.100	0.033	-48	104	-105	229
425	0.090	0.030	-43	94	-95	207
450	0.081	0.027	-39	85	-86	188
475	0.074	0.025	-36	78	-79	171
500	0.068	0.023	-33	71	-72	156
525	0.062	0.021	-30	65	-66	143
550	0.057	0.019	-28	60	-61	131
575	0.053	0.018	-25	55	-56	121
600	0.047	0.017	-23	51	-52	112

Table 4.18: Calculated Stresses and Strains at the bottom of Cement Stabilized Base Layer

Depth (Z) (mm)	Soil-Cement: E_2 = 3000Mpa				E_2 = 750Mpa Strains ($^{-6}$x10mm/mm)	
	Stresses (N/mm^2)		Strains ($^{-6}$x10mm/mm)			
	σ_z	σ_H	ϵ_t	ϵ_z	ϵ_t	ϵ_z
225	0.231	-0.062	-39	88	-160	370
250	0.202	-0.058	-35	77	-140	320
275	0.178	-0.053	-31	68	-130	290
300	0.157	-0.048	-27	60	-120	250
325	0.139	-0.044	-24	53	-100	230
350	0.124	-0.040	-22	48	-90	200
375	0.111	-0.036	-20	42	-80	180
400	0.100	-0.033	-17	38	-70	160
425	0.090	-0.030	-16	34	-70	150
450	0.081	-0.027	-14	31	-60	130
475	0.074	-0.025	-13	28	-60	120
500	0.068	-0.023	-12	26	-50	110
525	0.062	-0.021	-11	24	-50	100
550	0.057	-0.019	-10	22	-40	90
575	0.053	-0.018	-9	20	-40	90
600	0.049	-0.017	-8	19	-40	80

Table 4.19: Calculated Stresses and Strains at the bottom of Cement Stabilized Base Layer

Soil-Cement E_2 = 4,500Mpa				
Depth (Z) (mm)	Stresses (N/mm^2)		Strains ($^{-6}$x10mm/mm)	
	σ_z	σ_H	ϵ_t	ϵ_z
225	0.231	-0.062	-30	60
250	0.202	-0.058	-20	50
275	0.178	-0.053	-20	50
300	0.157	-0.048	-20	40
325	0.139	-0.044	-20	40
350	0.124	-0.040	-20	30
375	0.111	-0.036	-10	30
400	0.100	-0.033	-10	30
425	0.090	-0.030	-10	20
450	0.081	-0.027	-10	20
475	0.074	-0.025	-10	20
500	0.068	-0.023	-10	20
525	0.062	-0.021	-10	20
550	0.057	-0.019	-10	20
575	0.053	-0.018	-10	10
600	0.049	-0.017	-10	10

Table 4.20: Calculated Stresses and Compressive Strains at the top of Subgrade

Depth (Z) (mm)	Stresses (N/mm^2)		Compressive $E_4 = 100Mpa$	Strains (*x10mm/mm) $E_4 = 200Mpa$
	σ_Z	σ_H	$\in_s$	$\in_c$
425	0.084	0.032	1094	547
450	0.076	0.029	992	496
475	0.069	0.026	903	451
500	0.063	0.024	825	412
525	0.058	0.022	756	378
550	0.053	0.021	695	348
575	0.049	0.019	641	321
600	0.045	0.018	593	297
625	0.042	0.016	550	275
650	0.039	0.015	511	256
675	0.036	0.014	476	238
700	0.034	0.013	445	223
725	0.032	0.012	417	208
750	0.030	0.012	390	195
775	0.028	0.011	367	184
800	0.033	0.010	345	173

Table 4.21: Calculated Stresses and Strains at the bottom of Lime Stabilized Base Layer

Depth (Z) (mm)	Soil-Lime Stresses (N/mm^2)		E_2 = 1100Mpa Strains (^{6}x10mm/mm)		E_2 = 750Mpa Strains (^{6}x10mm/mm)	
	σ_Z	σ_H	ϵ_t	ϵ_z	ϵ_t	ϵ_z
225	0.231	-0.062	-110	250	-160	370
250	0.202	-0.058	-100	220	-140	320
275	0.178	-0.053	-90	200	-130	290
300	0.157	-0.048	-80	170	-120	250
325	0.139	-0.044	-70	150	-100	230
350	0.124	-0.040	-60	140	-90	200
375	0.111	-0.036	-60	120	-80	180
400	0.100	-0.033	-50	110	-70	160
425	0.090	-0.030	-50	100	-70	150
450	0.081	-0.027	-40	90	-60	130
475	0.074	-0.025	-40	80	-60	120
500	0.068	-0.023	-40	80	-50	110
525	0.062	-0.021	-30	70	-50	100
550	0.057	-0.019	-30	60	-40	90
575	0.053	-0.018	-30	60	-40	90
600	0.047	-0.017	-30	50	-40	80

Table 4.22: Calculated Stresses and Strains at the bottom of Lime Stabilized Base Layer

Soil-Lime E_2 = 500Mpa				
Depth (Z) (mm)	Stresses (N/mm^2)		Strains ($^{-6}$x10mm/mm)	
	σ_z	σ_H	ϵ_t	ϵ_z
225	0.231	-0.062	-240	550
250	0.202	-0.058	-220	490
275	0.178	-0.053	-190	430
300	0.157	-0.048	-170	380
325	0.139	-0.044	-150	340
350	0.124	-0.040	-140	300
375	0.111	-0.036	-120	270
400	0.100	-0.033	-110	240
425	0.090	-0.030	-100	220
450	0.081	-0.027	-90	200
475	0.074	-0.025	-80	180
500	0.068	-0.023	-80	170
525	0.062	-0.021	-70	150
550	0.057	-0.019	-70	140
575	0.053	-0.018	-60	130
600	0.047	-0.017	-60	120

Table 4.23: Calculated Stresses and Strains at the bottom of Cement Stabilized Sub-Base

Depth (Z) (mm)	Soil-Cement Stresses (N/mm^2)		E_3 = 1100Mpa Strains ($^{-6}$x10mm/mm)		E_3 = 500Mpa Strains ($^{-6}$x10mm/mm)	
	σ_Z	σ_H	ϵ_t	ϵ_z	ϵ_t	ϵ_z
425	0.090	-0.030	-50	100	-100	220
450	0.081	-0.027	-40	90	-90	200
475	0.074	-0.025	-40	80	-80	180
500	0.068	-0.023	-40	80	-80	170
525	0.062	-0.021	-30	70	-70	150
550	0.057	-0.019	-30	60	-70	140
575	0.053	-0.018	-30	60	-60	130
600	0.049	-0.017	-30	50	-60	120
625	0.045	-0.015	-20	50	-50	110
650	0.042	-0.014	-20	50	-50	100
675	0.039	-0.013	-20	40	-40	100
700	0.036	-0.013	-20	40	-40	90
725	0.034	-0.012	-20	40	-40	80
750	0.032	-0.011	-20	40	-40	80
775	0.030	-0.010	-20	30	-30	70
800	0.028	-0.010	-10	30	-30	70

Table 4.24: Calculated Stresses and Strains at the bottom of Cement Stabilized Sub-Base

Depth (Z) (mm)	Soil-Cement Stresses (N/mm^2)		E_3 = 3000Mpa Strains ($^{-6}$x10mm/mm)		E_3 = 750Mpa Strains ($^{-6}$x10mm/mm)	
	σ_z	σ_H	ϵ_t	ϵ_z	ϵ_t	ϵ_z
425	0.090	-0.030	-20	40	-70	150
450	0.081	-0.027	-20	30	-60	130
475	0.074	-0.025	-10	30	-60	120
500	0.068	-0.023	-10	30	-50	110
525	0.062	-0.021	-10	30	-50	100
550	0.057	-0.019	-10	20	-40	90
575	0.053	-0.018	-10	20	-40	90
600	0.049	-0.017	-10	20	-40	80
625	0.045	-0.015	-10	20	-30	70
650	0.042	-0.014	-10	20	-30	70
675	0.039	-0.013	-10	20	-30	60
700	0.036	-0.013	-10	20	-30	60
725	0.034	-0.012	-10	10	-30	60
750	0.032	-0.011	-10	10	-20	50
775	0.030	-0.010	-10	10	-20	50
800	0.028	-0.010	-10	10	-20	50

Table 4.25: Calculated Stresses and Strains at the bottom of Lime Stabilized Sub-Base

Depth (Z) (mm)	Soil-Lime Stresses (N/mm^2)		E_3 = 1100Mpa Strains ($^{-6}$x10mm/mm)		E_3 = 750Mpa Strains ($^{-6}$x10mm/mm)	
	σ_z	σ_{tt}	$\in_t$	$\in_z$	$\in_t$	$\in_z$
425	0.090	-0.030	-50	100	-70	150
450	0.081	-0.027	-40	90	-60	130
475	0.074	-0.025	-40	80	-60	120
500	0.068	-0.023	-40	80	-50	110
525	0.062	-0.021	-30	70	-50	100
550	0.057	-0.019	-30	60	-40	90
575	0.053	-0.018	-30	60	-40	90
600	0.049	-0.017	-30	50	-40	80
625	0.045	-0.015	-20	50	-30	70
650	0.042	-0.014	-20	50	-30	70
675	0.039	-0.013	-20	40	-30	60
700	0.036	-0.013	-20	40	-30	60
725	0.034	-0.012	-20	40	-30	60
750	0.032	-0.011	-20	40	-20	50
775	0.030	-0.010	-20	30	-20	50
800	0.028	-0.010	-10	30	-20	50

Table 4.26: Calculated Stresses and Strains at the bottom of Lime Stabilized Sub-Base

Depth (Z) (mm)	Soil-Cement Stresses (N/mm^2)		E_3 = 500Mpa Strains ($^{-6}$x10mm/mm)	
	σ_z	σ_{rr}	$\in_t$	$\in_z$
425	0.090	-0.030	-100	220
450	0.081	-0.027	-90	200
475	0.074	-0.025	-80	180
500	0.068	-0.023	-80	170
525	0.062	-0.021	-70	150
550	0.057	-0.019	-70	140
575	0.053	-0.018	-60	130
600	0.049	-0.017	-60	120
625	0.045	-0.015	-50	110
650	0.042	-0.014	-50	100
675	0.039	-0.013	-40	100
700	0.036	-0.013	-40	90
725	0.034	-0.012	-40	80
750	0.032	-0.011	-40	80
775	0.030	-0.010	-30	70
800	0.028	-0.010	-30	70

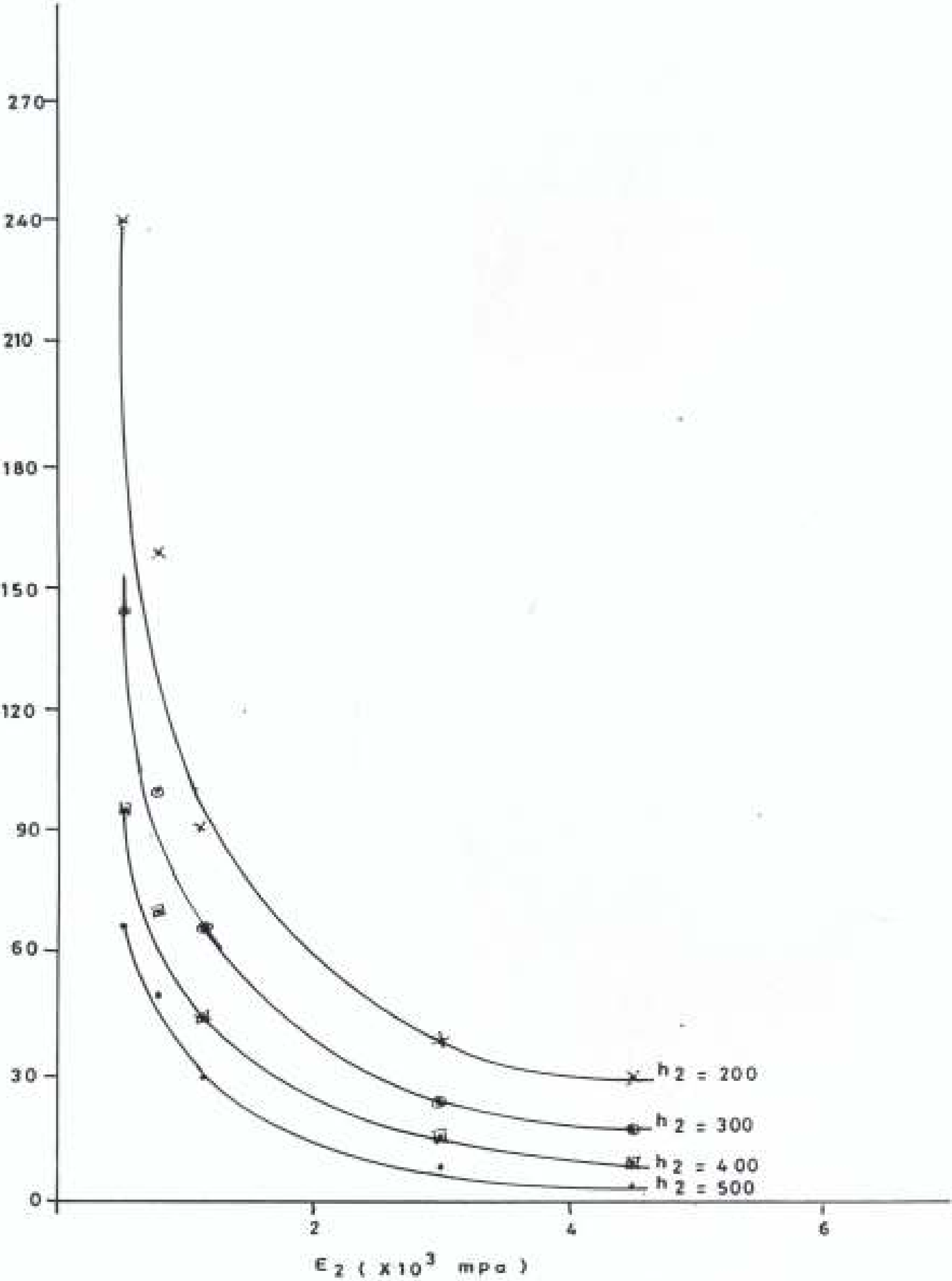


Fig. 4·08 Effects of base thickness on the tensile strain at the bottom of stabilized bases.

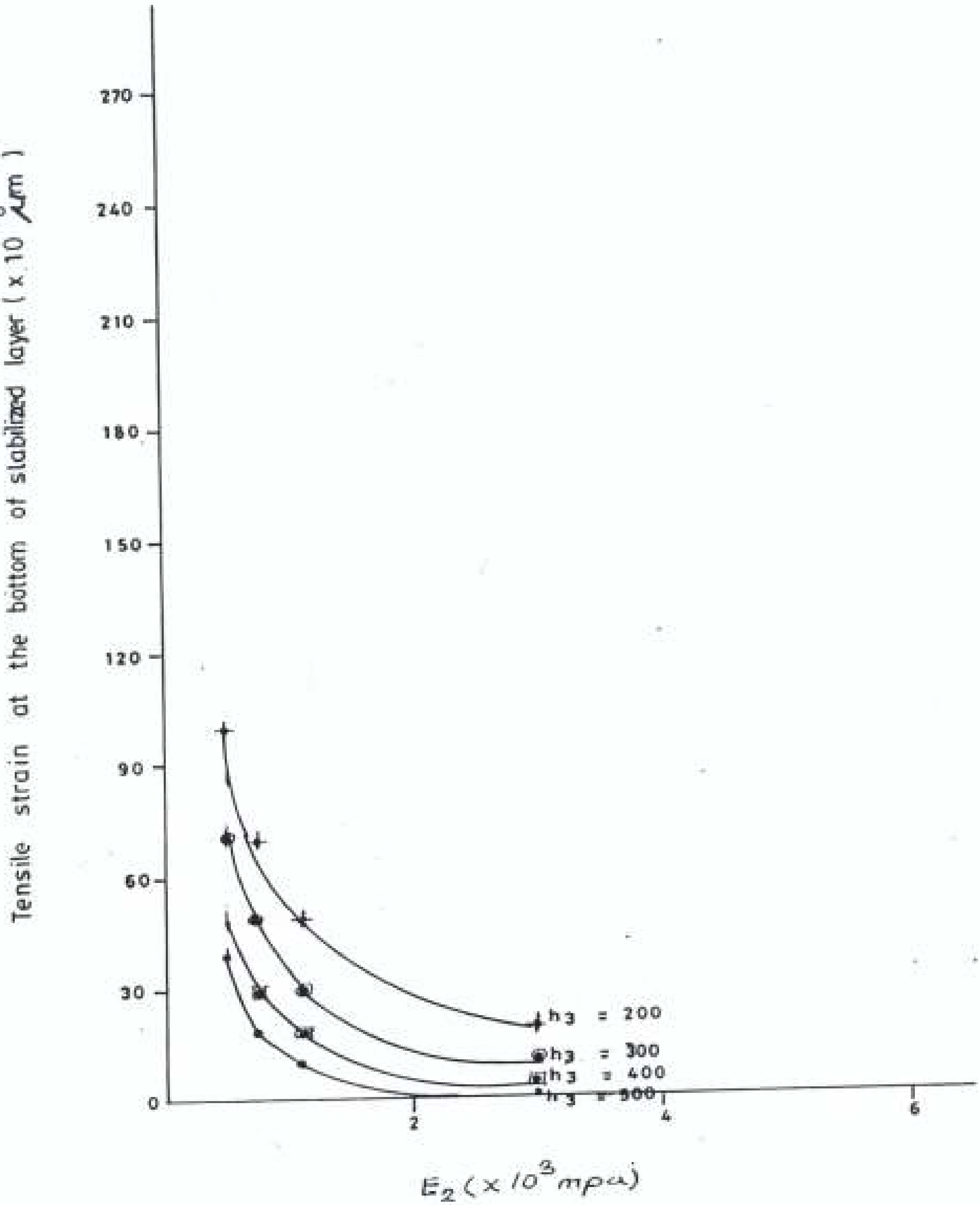


Fig. 4·09 Effects of sub-base thickness on the tensile strain at the bottom of stabilized sub-base.

of the layers. This fact is obvious from figure 4.09 where the strains at the bottom of treated sub-base is less than those at the bottom of treated base for the same overall thickness. This is so because stabilized sub-basecourse is by location at a lower level than the basecourse for every thickness.

Tables 4.27 to 4.46 show the calculated strains compared with the allowable strains at the critical points of the pavement for asphaltic surfacing, soil-cement base and subgrade. These calculations are based on different moduli of these layers for both no-cracking and cracking criteria. Soil-cement has different strain at break (ϵ_l) for different moduli and thus different allowable strains. Four strain values are picked for analysis per layer. For cracking criterion, the values for soil-cement and subgrade are left as calculated, but for the no-cracking criterion, the values for soil-cement and subgrade are multiplied by 1.2 and 3.5 respectively. This is so because according to Wilson and Pretorius (1970) who used prismatic solid finite element analysis, local increases in the vertical strain immediately beneath the cracks in treated layers are significant and could be up to 14 times of initial vertical strain Table 4.35 shows what they suggested as increases in calculated maximum strains to accommodate initital cracking for different thicknesses of treated layer.

The strain values from tables 4.27 to 4.34 give rise to preliminary thicknesses of the pavements. It is seen also from the tables that the allowable strain for cement treated base changes with changes in the modulus of the layer. The number of load repetitions used for arriving at these allowable values are shown on the tables. In tables 4.29 and 4.33, the calculated compressive strains

Table 4.27: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 100Mpa for Cracking Criteria

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t(\mu\in)$	Allowable Strain $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 36	200		
Cement-treated base	5×10^5	146, 130, 117, 105	154		
Subgrade	1.362×10^5 (E80)			756, 695, 641, 593,	969

Table 4.28: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 200Mpa for Cracking Criteria

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t(\mu\in)$	Allowable Strains $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 36	200		
Cement-treated base	5×10^3	146, 130, 117, 105	154		
Subgrade	1.362×10^5 (E80)			378, 348, 321, 297,	969

Table 4.29: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 100Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t(\mu\in)$	Allowable Strain $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 36	200		
Cement-treated base	5×10^3	48, 48, 36, 36	154		
Subgrade	1.362×10^5 (E80)			1460, 1365, 1285, 1208,	969

Table 4.30: Calculated Versus Allowable Strains in Pavement Layout for E_2 = 500Mpa and E_4 = 200Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\epsilon_t(\mu\epsilon)$	Allowable Strains $\epsilon_t(\mu\epsilon)$	Calculated Strains $\epsilon_c(\mu\epsilon)$	Allowable Strain $\epsilon_c(\mu\epsilon)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Cement-treated base	5 x 10^3	113, 102, 94, 85	154		
Subgrade	1.362 x 10^5 (E80)			963, 896, 838, 781,	969

Table 4.31: Calculated Versus Allowable Strains in Pavement Layout for E_2 = 1100Mpa and E_4 = 100Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\epsilon_t(\mu\epsilon)$	Allowable Strain $\epsilon_t(\mu\epsilon)$	Calculated Strains $\epsilon_c(\mu\epsilon)$	Allowable Strain $\epsilon_c(\mu\epsilon)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Cement-treated base	5 x 10^3	84, 74, 66, 59	95		
Subgrade	1,362 x 10^5 (E80)			903, 825, 756, 695,	969

Table 4.32: Calculated Versus Allowable Strains in Pavement Layout for E_2 = 1100Mpa and E_4 = 200Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\epsilon_t(\mu\epsilon)$	Allowable Strains $\epsilon_t(\mu\epsilon)$	Calculated Strains $\epsilon_c(\mu\epsilon)$	Allowable Strain $\epsilon_c(\mu\epsilon)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Cement-treated base	5 x 10^3	84, 74, 66, 59	95		
Subgrade	1.362 x 10^5 (E80)			451, 412, 378, 348,	969

Table 4.33: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 100Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\epsilon_t(\mu\epsilon)$	Allowable Strain $\epsilon_t(\mu\epsilon)$	Calculated Strains $\epsilon_c(\mu\epsilon)$	Allowable Strain $\epsilon_c(\mu\epsilon)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Cement-treated base	5 x 10^3	36, 34, 30, 28	95		
Subgrade	1.362 x 10^5 (E80)			1460, 1365, 1285, 1208,	969

Table 4.34: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 200Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\epsilon_t(\mu\epsilon)$	Allowable Strains $\epsilon_t(\mu\epsilon)$	Calculated Strains $\epsilon_c(\mu\epsilon)$	Allowable Strain $\epsilon_c(\mu\epsilon)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Cement-treated base	5 x 10^3	52, 47, 43, 40	95		
Subgrade	1.362 x 10^5 (E80)			962, 896, 833, 781,	969

Table 4.35: Suggested Increases in Calculated Maximum Strains to Accommodate Initial Cracking

Type of Cracking	Total Thickness Treated (mm)	Max. Horizontal Tensile Strain	Max. Vertical Strain on Top of Layers Below Cracked Layer	
			First Underlying Layer	Second Underlying Layer
No-cracking expected (e.g. < 2% cement or lime)		1.0	1.0	1.0
Moderate cracking; crack width < 2mm (e.g. natural material with 2 - 3% cement or lime)	≤ 200 > 200	1.10 1.20	2.5 7.0	1.5 3.5
Extensive cracking; crack width > 2mm (e.g. crushed stone with 4 - 6% cement	≤ 200 > 200	1.25 1.40	5.0 14.0	2.5 7.0

Source: Otte (1978)

Table 4.36: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 100Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_i)	Calculated Strains $\in_t(\mu\in)$	Allowable Strain $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated base	10^6	140, 120, 110, 100,	140		
Subgrade	1.362 x 10^5 (E80)			695, 641, 593, 550,	969

Table 4.37: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 200Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_i)	Calculated Strains $\in_t(\mu\in)$	Allowable Strains $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated base	10^6	140, 120, 110, 100,	140		
Subgrade	1.362 x 10^5 (E80)			348, 321, 297, 275,	969

Table 4.38: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 100Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_i)	Calculated Strains $\in_t(\mu\in)$	Allowable Strain $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated base	10^6	84, 84, 72, 72	140		
Subgrade	1.362 x 10^5 (E80)			1460, 1365, 1285, 1208,	969

Table 4.39: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 200Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\epsilon_t (\mu\epsilon)$	Allowable Strains $\epsilon_t (\mu\epsilon)$	Calculated Strains $\epsilon_c (\mu\epsilon)$	Allowable Strain $\epsilon_c (\mu\epsilon)$
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Lime treated base	10^6	120, 108, 96, 96	140		
Subgrade	1.362×10^5 (E80)			963, 896, 833, 781,	969

Table 4.40: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 100Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\epsilon_t (\mu\epsilon)$	Allowable Strain $\epsilon_t (\mu\epsilon)$	Calculated Strains $\epsilon_c (\mu\epsilon)$	Allowable Strain $\epsilon_c (\mu\epsilon)$
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Lime treated base	10^6	90, 80, 70, 60	140		
Subgrade	1.362×10^5 (E80)			903, 825, 756, 695,	969

Table 4.41: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 200Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\epsilon_t (\mu\epsilon)$	Allowable Strains $\epsilon_t (\mu\epsilon)$	Calculated Strains $\epsilon_c (\mu\epsilon)$	Allowable Strain $\epsilon_c (\mu\epsilon)$
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Lime treated base	10^6	90, 80, 70, 60	140		
Subgrade	1.362×10^5 (E80)			451, 412, 378, 348,	969

Table 4.42: Calculated Versus Allowable Strains in Pavement Layout for E_2 = 1100Mpa and E_4 = 100Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t(\mu\in)$	Allowable Strain $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated base	10^6	36, 36, 36, 36	140		
Subgrade	1.362 x 10^5 (E80)			1460, 1365, 1285, 1208,	969

Table 4.43: Calculated Versus Allowable Strains in Pavement Layout for E_2 = 1100Mpa and E_4 = 200Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t(\mu\in)$	Allowable Strains $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated base	10^6	60, 48, 48, 48	140		
Subgrade	1.362 x 10^5 (E80)			963, 896, 833, 781,	969

Table 4.44: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 100Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t(\mu\in)$	Allowable Strain $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_c(\mu\in)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Cement treated Sub-base	5 x 10^3	80, 80, 70, 70	154		
Subgrade	1.362 x 10^5 (E80)			903, 896, 833, 781,	969

Table 4.45: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 200Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t(\mu\in)$	Allowable Strains $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_d(\mu\in)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Cement treated Sub-base	5 x 10^3	80, 80, 70, 70	154		
Subgrade	1.362 x 10^5 (E80)			451, 412, 378, 348,	969

Table 4.46: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 100Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t(\mu\in)$	Allowable Strain $\in_t(\mu\in)$	Calculated Strains $\in_c(\mu\in)$	Allowable Strain $\in_d(\mu\in)$
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 36	200		
Cement treated Sub-base	5 x 10^3	48, 48, 36, 36	154		
Subgrade	1.362 x 10^5 (E80)			1460, 1365, 1285, 1208,	969

$\mu\in = \times 10^{-6}$ mm/mm

exceed the allowable strain. Tables 4.36 to 4.43 show the calculated versus allowable strains for the asphaltic surfacing, soil-lime basecourse and the subgrade for different stiffnesses of the layers. One of the major differences between tables 4.27 to 4.34 and tables 4.36 to 4.43 is that the allowable strains of soil-cement in the first set of tables changes different moduli whereas the allowable strains of the soil-lime in the second set of tables remain constant. Four strain values were picked in each case for the surfacing, treated base and subgrade and all these define the preliminary thicknesses of the pavement layers.

From all these tables for soil-cement and soil-lime, it is observed that the calculated tensile strain values for the weaker subgrade (100Mpa) and the stronger one (200Mpa) are the same for cracking criterion. Meanwhile, for the same condition, the calculated compressive strains for the stronger subgrade is far less than those of the weaker one. This implies that for the same tensile strain values at the bottom of treated layers, a pavement with a higher stiffness is less prone to distress than the one with a lower stiffness. In the case of no-cracking criterion, though the strain values at the bottom of treated base is less for a weaker subgrade (100Mpa), all the calculated compressive strains at the top of subgrade are more than the allowable strain and this implies that the subgrade will in no time deform when and if cracks are initiated in the pavement. The factors suggested by Wilson and Pretorius (1970) is more or less a factor of safety to prevent or pre-empt cracks initiation by designing for heavier loads. General observation of the tables reveals that tensile strain values for cracking

Table 4.47: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 200Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains ϵ_t ($\mu\epsilon$)	Allowable Strains ϵ_t ($\mu\epsilon$)	Calculated Strains ϵ_c ($\mu\epsilon$)	Allowable Strain ϵ_c ($\mu\epsilon$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Cement treated Sub-base	5×10^5	60, 60, 48, 48,	154		
Subgrade	1.362×10^5 (E80)			963, 896, 838, 781,	969

Table 4.48: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 100Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains ϵ_t ($\mu\epsilon$)	Allowable Strain ϵ_t ($\mu\epsilon$)	Calculated Strains ϵ_c ($\mu\epsilon$)	Allowable Strain ϵ_c ($\mu\epsilon$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Cement treated Sub-base	5×10^5	24, 24, 24, 12,	95		
Subgrade	1.362×10^5 (E80)			1460, 1365, 1285, 1208,	969

Table 4.49: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 200Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains ϵ_t ($\mu\epsilon$)	Allowable Strains ϵ_t ($\mu\epsilon$)	Calculated Strains ϵ_c ($\mu\epsilon$)	Allowable Strain ϵ_c ($\mu\epsilon$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Cement treated Sub-base	5×10^5	24, 24, 24, 24,	95		
Subgrade	1.362×10^5 (E80)			963, 896, 833, 781,	969

Table 4.50: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 100Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t$ ($\mu\in$)	Allowable Strain $\in_t$ ($\mu\in$)	Calculated Strains $\in_c$ ($\mu\in$)	Allowable Strain $\in_c$ ($\mu\in$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Cement treated Sub-base	5×10^5	40, 40, 30, 30,	95		
Subgrade	1.362×10^5 (E80)			903, 825, 756, 695,	969

Table 4.51: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 200Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t$ ($\mu\in$)	Allowable Strains $\in_t$ ($\mu\in$)	Calculated Strains $\in_c$ ($\mu\in$)	Allowable Strain $\in_c$ ($\mu\in$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Cement treated Sub-base	5×10^5	50, 40, 40, 40,	95		
Subgrade	1.362×10^5 (E80)			547, 451, 412, 378,	969

Table 4.52: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 100Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t$ ($\mu\in$)	Allowable Strain $\in_t$ ($\mu\in$)	Calculated Strains $\in_c$ ($\mu\in$)	Allowable Strain $\in_c$ ($\mu\in$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Lime treated Sub-base	10^6	80, 80, 70, 70	140		
Subgrade	1.362×10^5 (E80)			903, 825, 756, 695,	969

Table 4.53: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 200Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t$ ($\mu\in$)	Allowable Strains $\in_t$ ($\mu\in$)	Calculated Strains $\in_c$ ($\mu\in$)	Allowable Strain $\in_c$ ($\mu\in$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Lime treated Sub-base	10^6	80, 80, 70, 70	140		
Subgrade	1.362×10^5 (E80)			451, 412, 378, 348,	969

Table 4.54: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 100Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t$ ($\mu\in$)	Allowable Strain $\in_t$ ($\mu\in$)	Calculated Strains $\in_c$ ($\mu\in$)	Allowable Strain $\in_c$ ($\mu\in$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Lime treated Sub-base	10^6	48, 48, 36, 36	140		
Subgrade	1.362×10^5 (E80)			1460, 1365, 1285, 1208,	969

Table 4.55: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 500Mpa and E_4 = 200Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t$ ($\mu\in$)	Allowable Strains $\in_t$ ($\mu\in$)	Calculated Strains $\in_c$ ($\mu\in$)	Allowable Strain $\in_c$ ($\mu\in$)
Asphaltic Concrete Surfacing	5×10^6	4, 19, 30, 35	200		
Lime treated Sub-base	10^6	60, 60, 48, 48	140		
Subgrade	1.362×10^5 (E80)			963, 896, 833, 781,	969

Table 4.56: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 100Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t$ ($\mu\in$)	Allowable Strain $\in_t$ ($\mu\in$)	Calculated Strains $\in_c$ ($\mu\in$)	Allowable Strain $\in_c$ ($\mu\in$)
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated Sub-base	10^6	40, 40, 30, 30	140		
Subgrade	1.362 x 10^5 (E80)			903, 825, 756, 695,	969

Table 4.57: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 200Mpa for Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains $\in_t$ ($\mu\in$)	Allowable Strains $\in_t$ ($\mu\in$)	Calculated Strains $\in_c$ ($\mu\in$)	Allowable Strain $\in_c$ ($\mu\in$)
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated Sub-base	10^6	40, 40, 30, 30	140		
Subgrade	1.362 x 10^5 (E80)			451, 412, 378, 348,	969

Table 4.58: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 100Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains ϵ_t ($\mu\epsilon$)	Allowable Strain ϵ_t ($\mu\epsilon$)	Calculated Strains ϵ_c ($\mu\epsilon$)	Allowable Strain ϵ_c ($\mu\epsilon$)
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated Sub-base	10^6	24, 24, 24, 12	140		
Subgrade	1.362 x 10^5 (E80)			1460, 1365, 1285, 1208,	969

Table 4.59: Calculated Versus Allowable Strains in Pavement Layout for E_3 = 1100Mpa and E_4 = 200Mpa for No-Cracking Criterion

Pavement Layer	Number of Load Repetitions (N_f)	Calculated Strains ϵ_t ($\mu\epsilon$)	Allowable Strains ϵ_t ($\mu\epsilon$)	Calculated Strains ϵ_c ($\mu\epsilon$)	Allowable Strain ϵ_c ($\mu\epsilon$)
Asphaltic Concrete Surfacing	5 x 10^6	4, 19, 30, 35	200		
Lime treated Sub-base	10^6	24, 24, 24, 24	140		
Subgrade	1.362 x 10^5 (E80)			963, 896, 833, 781,	969

criterion is usually less than those of no-cracking criterion. The interpretation of this is that designs for no-cracking criterion is more conservative in terms of pavement layer thicknesses and thus pavement materials.

The calculated versus allowable strains for the asphaltic surfacing, treated sub-basecourse and subgrade are shown in Tables 4.44 to 4.59. Tables 4.44 to 4.53 show calculated strain values for cement-treated sub-base while tables 4.54 to 4.59 show the strain values for lime treated sub-base. The pattern in the previous tables regarding the decreasing values of strains as the depth increase is the same with these tables too. An obvious difference, which is expected, is that the tensile strain values at the bottom of treated sub-base is less than those for the basecourse. The tensile strains at the bottom of asphaltic concrete surfacing and the compressive strain at the top of subgrade are more or less the same with those of treated basecourses. Different stiffnesses used for the sub-base and subgrade resulted into different allowable strains for cement treated while allowable strain value for soil-lime is constant. It is also observed from the tables that 25mm (an inch) or 50mm (2 inches) increase in depth at times does not have effect on the tensile strain values at the bottom of a treated sub-base.

4.7 COMPUTER PROGRAM AND OUTPUT

The outputs of a FORTRAN Computer Program for BISAR Programme equations are shown in tables 6.08 in Appendix III. The program combines Peatie and Ullidtz (1982) equations with BISAR Programme equation. Peatie and Ullidtz (1982) equations did not give consistent results and therefore the outputs are on BISAR Programme only. The computer program was written to calculate stresses and strains at the bottom of asphaltic concrete layer, at the bottom of soil-cement and soil-lime layers and at the top of subgrade. The outputs of this programme had been simplified and applied in Tables 4.17 to 4.59.

4.8 RESULTS OF OPTIMAL MATERIAL COMBINATIONS AND LAYER THICKNESSES

Tables 4.27 to 4.34 and 4.36 to 4.59 show four values of strains for asphaltic concrete surfacing, treated basecourse, sub-basecourse and subgrade for different moduli of these materials which satisfy the maximum permissible strain values of each of the materials. The tables also show different load repetitions which account for the allowable strains in each case. Because the most critical factors in treated layers are the tensile strain at the bottom of the treated layers and the compressive strain at the top of the subgrade, the strain values (four in each case) for the treated layers and the subgrade define the preliminary material combinations and layer thicknesses. For brevity, the values arising from 1100Mpa modulus are used for soil-cement basecourse and sub-basecourse while those arising from 500Mpa modulus are used for soil-lime base-course and 1100Mpa for sub-basecourse.

Relating the strain values in table 6.01 in Appendix to the pavement layout for E_4 = 100Mpa and E_4 = 200Mpa in the same table, we have the total pavement section (H) as 475mm, 500mm, 525mm and 550mm for E_4 = 100Mpa and 425mm, 450mm, 475mm and 500mm for E_4 = 200Mpa.

Using the adapted Hejal et al (1971) minimization equation, the optimal pavement layout is 625mm consisting of:-

1.	Asphalt Surfacing (h_1)	=	100mm
2.	Soil-Cement Base (h_2)	=	325mm
3.	Laterite Sub-Base (h_3)	=	200mm
			625mm

These material combinations and layer thicknesses are the most efficient for a subgrade with soaked CBR of 5% and modulus of 100Mpa. The analysis is shown in Appendix II. The cost per square metre of this combination is also estimated to be₦ 854.87. When the soaked CBR changes to 10% with E_4 constant, the layout becomes 600mm consisting of -

1.	Asphalt Surfacing (h_1)	=	100mm
2.	Soil-Cement Base (h_2)	=	300mm
3.	Laterite Sub-Base (h_3)	=	200mm
			600mm

The estimated cost per square metre is₦ 831.16. For the subgrade condition in which E_4 = 200Mpa and soaked CBR = 5%, the layout becomes 625mm comprising of -

1. Asphalt Surfacing (h_1) = 100mm

2. Soil-Cement Base (h_2) = 325mm

3. Laterite Sub-Base (h_3) = 200mm
   625mm

The results of this optimization process are shown in Tables 4.60 to 4.65. It is observed from table 4.60 that the optimal layout value is generally more than any of the preliminary layout values. This is so because all the values obtained as preliminary layout are theoretical in nature and do not account to a large extent for the practical situation of the proposed site. The inclusion of serviceability index, soil support values, subgrade condition as regional factor help in making the pavement layout more realistic as it is shown by optimal layout values. As an example, it is seen from table 4.60 that a sub-base with a lower CBR value has a higher optimal value. This is why it is observed from the table that a CBR of 5% has an optimal layout of 625mm compared to 600mm for a CBR of 10%. Table 4.60 also shows that the subgrade modulus values have effects on the values of preliminary layouts. These effects are however neutralized by the condition of sub-base and this is the reason why the optimal value for E_4 = 100Mpa and 5% CBR and E_4 = 200Mpa and 5% CBR are the same. The same thing is true for E_4 = 100Mpa and 5% CBR with E_4 = 200Mpa and 5%. This implies that, although, subgrade condition has an insert tables 4.60 - 4.65

Table 4.60: Preliminary Versus Optimal Layout of Pavement Sections for Soil-Cement Base with Respect to Cracking Criterion

Subgrade Modulus E_4	Sub-base CBR (%)	Base Modulus E_2	Preliminary Layout (mm)	Optimal Layout (mm)	Optimal Cost (₦/m²)
100	5(Soaked)	1100	475 500 525 550	h_1 = 100 625 h_2 = 325 h_3 = 200	854.87
100	10(Soaked)	1100	475 500 525 550	h_1 = 100 600 h_2 = 300 h_3 = 200	831.16
200	5(Soaked)	1100	425 450 475 500	h_1 = 100 625 h_2 = 325 h_3 = 200	854.87
200	10(Soaked)	1100	425 450 475 500	h_1 = 100 600 h_2 = 300 h_3 = 200	831.16

Table 4.61: Preliminary Versus Optimal Layout of Pavement Sections for Soil-Lime Base with Respect to Cracking Criterion

Subgrade Modulus E_4	Sub-base CBR (%)	Base Modulus E_2	Preliminary Layout (mm)	Optimal Layout (mm)	Optimal Cost (₦/m²)
100	5(Soaked)	500	550 575 600 625	h_1 = 50 550 h_2 = 300 h_3 = 200	625.24
100	10(Soaked)	500	550 575 600 625	h_1 = 50 550 h_2 = 300 h_3 = 200	625.24
200	5(Soaked)	500	550 575 600 625	h_1 = 25 550 h_2 = 325 h_3 = 200	561.22
200	10(Soaked)	500	550 575 600 625	h_1 = 25 550 h_2 = 325 h_3 = 200	561.22

Table 4.62: Preliminary Versus Optimal Layout of Pavement Sections for Soil-Cement Base with Respect to No-Cracking Criterion

Subgrade Modulus E_4	Sub-base CBR (%)	Base Modulus E_2	Preliminary Layout (mm)	Optimal Layout (mm)	Optimal Cost (₦/m²)
200	5(Soaked)	1100	625 650 675 700	h_1 = 100 625 h_2 = 325 h_3 = 200	854.87
200	10(Soaked)	1100	625 650 675 700	h_1 = 100 625 h_2 = 325 h_3 = 200	854.87

Table 4.63: Preliminary Versus Optimal Layout of Pavement Sections for Soil-Lime Base with Respect to No-Cracking Criterion

Subgrade Modulus E_4	Sub-base CBR (%)	Base Modulus E_2	Preliminary Layout (mm)	Optimal Layout (mm)	Optimal Cost (₦/m²)
200	5(Soaked)	500	625 650 675 700	h_1 = 75 625 h_2 = 350 h_3 = 200	774.40
200	10(Soaked)	500	625 650 675 700	h_1 = 75 625 h_2 = 350 h_3 = 200	774.40

Table 4.64: Preliminary Versus Optimal Layout of Pavement Sections for Soil-Cement Sub-base with Respect to Cracking and No-Cracking Criteria

Criteria	Subgrade Moduli E_4	Sub-Base Modulus E_3	Base CBR (%)	Preliminary Layout (mm)	Optimal Layout (mm)	Optimal Cost (₦/m²)
Cracking	100 & 200	1100	78(Unsoaked)	475 500 525 550	h_1 = 100 525 h_3 = 425	703.34
No-Cracking	200	1100	78(Unsoaked)	625 650 675 700	h_1 = 100 625 h_3 = 525	771.98

Table 4.65: Preliminary Versus Optimal Layout of Pavement Sections for Soil-Lime Sub-base with Respect to Cracking and No-Cracking Criteria

Criteria	Subgrade Moduli E_4	Base Modulus E_3	Base CBR (%)	Preliminary Layout (mm)	Optimal Layout (mm)	Optimal Cost (₦/m²)
Cracking	100 & 200	1100	78(Unsoaked)	425 450 475 500	h_1 = 75 425 h_3 = 350	549.34
No-Cracking	200	1100	78(Unsoaked)	625 650 675 700	h_1 = 100 625 h_3 = 525	771.98

important role to play in pavement design, other layers of the pavement also have their contributory roles. Another reason why the optimal layout is more than the preliminary layout values is that soil-cement used here has a weak layer co-efficient (a_i) because of its lower compressive strength. A layer coefficient converts the structural number of a pavement to actual pavement thickness. It follows therefore, that given the same structural number, a smaller layer coefficient will result into a larger pavement thickness. Another reason for the larger values of optimal layout is obedience to constraints equations 6.08 to 6.19 in Appendix II.

Table 4.61 shows the optimal layout of soil-lime base. It is observed from the table that the optimal layout value falls within the preliminary layout values. This is probably so because soil-lime has a higher layer coefficient value. The layer coefficient values for soil-cement and soil-lime are 0.14 and 0.27 respectively. The soil-cement layer coefficient is about 52% of the layer coefficient for soil-lime. In addition, comparing Tables 4.60 and 4.61, the material combination for soil-lime is better than that of soil-cement. The reason still revolves around layer coefficient value for soil-lime.

It is noteworthy that the base modulus (E_z) under which a smaller optimal layout is obtained for soil-lime is less than that of soil-cement. All these tables show the optimal layout of pavement when considering cracking as a criterion.

Table 4.62 shows the preliminary and optimal layout of soil-cement base for no-cracking criterion. It is seen from the Table that the optimal layout is the same with that of cracking criterion. Also, a close look at the preliminary layout values of the cracking criterion reveals that the optimal layout for the criterion is more than any of the preliminary values. The implication of this is that the optimal layout value (625mm) is a realistic value. It is also observed from the table that a change in the CBR condition from 5% to 10% does not affect it. Table 4.63 which shows the case of soil-lime base attests to this observation despite the fact that the modulus of subgrade has reduced from 1100Mpa to 500Mpa. Although, the material combination for soil-lime changes slightly and is more economical, the total optimal section (625mm) is still the same. This optimal layout value is probably a meeting value for both soil-cement and soil-lime bases for the conditions used for this study.

The preliminary versus optimal layout values for soil-cement and soil-lime sub-bases are shown by Tables 4.64 and 4.65 respectively. From the tables, it is seen that the laterite bases are excluded for the optimal layout values. The Hejal et al (1971) minimization equation rejects laterite bases for both soil-cement and soil-lime (See Appendix II item 6.21). This means that for the stated conditions in this study, what would have passed for an "upside down" pavement is moderated to a two-layer pavement by an optimization method. An "upside down" pavement consists of bituminous layer, a granular base-course and a cement or lime-treated sub-base. It is observed from Table 4.64 that the optimal layout for soil-cement in a three layer pavement structure is more than for this two-layer system for cracking criterion. The same pattern is also repeated in Table 4.65 for soil-lime. Meanwhile, for no-cracking criterion, the optimal layout for the two tables once again converge on a layout that seems to be the "optimal layout" of the optimal layouts. The implication of this is that whether a pavement structure is two or three layered, cracking or no-cracking, the following range of material combination and layer thicknesses are very important.

Asphalt surfacing (h_1)	=	75 to 100mm
Treated base (h_2)	=	325 to 350mm
Treated Sub-base h_3	=	350 to 525mm
Untreated Base or Sub-base (h_2 or h_3)	=	0 to 200mm

The material combination and total pavement thickness of any of this must add to 625mm.

Figure 4.10 shows the relationship between treated base, sub-base thicknesses and the asphalt layer for optimum layout of the pavement. For the treated base, a constant value of 200 mm (for laterite sub-base) is added to the values of asphalt and base picked from the figure. For the treated sub-base, no value will be added because the structure is two-layered. As an example, an asphaltic surfacing value of 75mm corresponds to a treated base of 350 plus a constant sub-base value of 200mm. This adds up to 625mm which is the optimal layout for soil-lime base. This same value without the sub-base constant value is equal to 425mm which is the optimal layout for soil-lime sub-base. For surfacing value of 100mm, the corresponding value of the treated base or sub-base is 325mm. Adding 200mm in case of three-layered structure, the pavement layout is 625mm which is the optimal layout for soil-cement base with 5% soaked CBR.

The figure works for both cracking and no-cracking criteria within the moduli investigated. From the figure, it is observed that any asphalt surfacing thickness